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8. Papers should average about 12,000 words in length and must be no longer than 18,000 words. As an approximation, each full page of typed text, table, or illustration is the equivalent of 300 words.

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coordinate system. It is demonstrated that the method of line equations offers protection against computational errors because of the check methods involved. No second check of the calculations is required.

DERIVATION OF LINE EQUATIONS

Before presenting actual problems, the derivation of the general line equations will be presented. In order to compute the equation of a line, the coordinate values of two points on the line must be known. In highway design, the values may be scaled from a map or they may be predetermined by limiting field conditions.

Fig. 1 illustrates the derivation of the general line equation $N = I \pm E \cot \theta$, in which N and E are the North and East coordinate values, respectively, of a point, and θ is the bearing angle of the line. The line AB is produced until it intersects the North-South axis to the coordinate grid. The intercept, I, is the distance from the origin, being plus when North and minus when South. The N value of any point A in terms of E, is $I \pm E \cot \theta$, hence the general equation, $N + I \pm E \cot \theta$. It is noted that the value of I may also be minus in sign.

COMPUTING A LINE EQUATION

Referring to Fig. 2, the method of computing a line equation is demonstrated. Suppose the coordinates of two points on a line are:

Point	N	E
A	237,429.631	205,229.498
B	236,771.389	204,780.262
Difference	658.242	449.236

The cotangent of the bearing angle $\theta = \dfrac{658.242}{449.236} = 1.4652477$

From trigonometric tables the angle $\theta = 34^\circ\ 18'\ 46''$ (interpolated to the nearest second). Referring to Fig. 1, the bearing of the line is southwest and the equation of the line is:

$$N = I + E \operatorname{cotan} \theta \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

Substituting the coordinate values of point A in Eq. 1, it becomes

$$237{,}429.631 = I + 1.4652477\ (205{,}229.498)$$

Solving,

$$I = -63{,}282.419$$

Substituting values in Eq. 1, it becomes

$$N = -63{,}282.419 + 1.4652477\ E \quad \ldots\ldots\ldots\ldots \quad (2)$$

In order to prove that no mistakes have been made in the computations the coordinates of point B are substituted in Eq. 2:

$$236{,}771.389 = -63{,}282.419 + 1.4652477\ (204{,}780.262)$$

Solving 236,771.389 = 236,771.389. The line equation has now been proven correct.

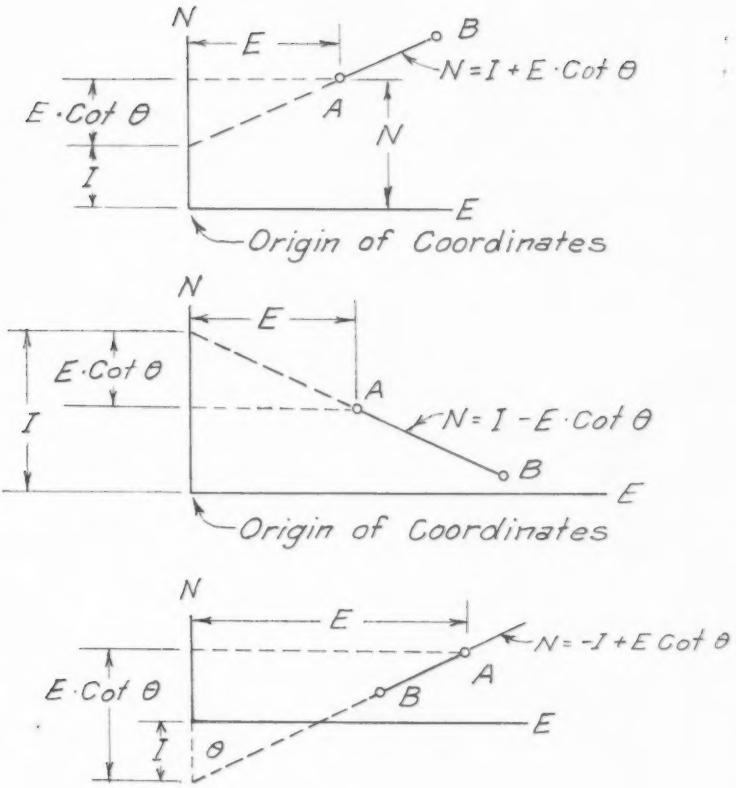

FIG. 1

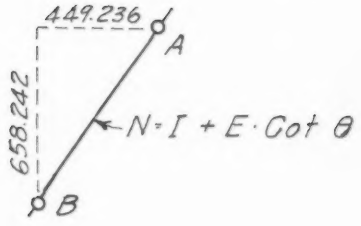

FIG. 2

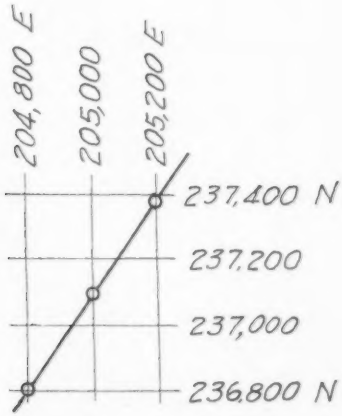

FIG. 3

PLOTTING A LINE

Let it be required to plot the line of Eq. 2. Choose three intersecting grid lines such as E 204,800, E 205,000, and E 205,200. By substituting these values in Eq. 2, the corresponding results are N 236,800.3, 237,093.4, and 237,386.4. If the three points fall on a straight line the computations and plotting are correct (Fig. 3).

Plotting in this manner is mathematically correct and will eliminate any error that may otherwise be caused by expansion or contraction of the drawing paper.

COMPUTATIONS FOR A SIMPLE HORIZONTAL CURVE

A very common problem in highway design is to connect two intersecting tangents with a simple horizontal curve. The coordinates of the point of intersection and any points on the curve can easily be found and proven by the line equation method.

The coordinates of the point of intersection, P.I., are found by substracting one equation from the other, thus eliminating N and solving for E. The value of E is then substituted in both equations and N will be found to be identical for both equations. If the value of N and E satisfy both equations the solution is correct and no further checking is required.

Example.—Given the equations of two intersecting lines, it is required to connect the two lines by a 3° curve and compute the coordinates of the point of intersection, P.I., point of curvature, P.C., point of tangency, P.T., and the center of the curve. The equations given are

$$N = -63{,}282.419 + 1.4652477\ E \quad \ldots\ldots\ldots\ldots \quad (3)$$

$$N = 124{,}348.889 + 0.5489909\ E \quad \ldots\ldots\ldots\ldots \quad (4)$$

The solution of this problem is as follows: By subtracting Eq. 4 from Eq. 3 we have:

$$0 = -187{,}631.318 + .9162568\ E$$

Solving, gives E = 204,780.262. This value is substituted in Eq. 3

$$N = -63{,}282.419 + 1.4652477\ (204{,}780.262) = 236{,}771.389$$

This value is substituted in Eq. 4

$$N = 124{,}348.889 + 0.5489909\ (204{,}780.262) = 236{,}771.389$$

The coordinates of the P.I. are

$$N = 236{,}771.389$$

$$E = 204{,}780.262$$

From the cotangent functions the bearing angles of the lines and the sin, cos, and tan, are tabulated as follows:

Angle:	Sin	Cos	Tan	Cotan
$\theta = 61°\ 14'\ 00''$	0.8765868	0.4812438	1.8215238	0.5489909
$\phi = 34°\ 18'\ 46''$	0.5637102	0.8259726	0.6823862	1.4652477

To ascertain the accuracy of the functions, the computations of the following relations should be performed:

$$\frac{\sin}{\cos} = \tan \text{ and } \frac{1}{\tan} = \text{cotan.}$$

Fig. 4 is referred to in making the computations.

Computations.

$$\begin{array}{r} 61^\circ\ 14'\ 00'' \\ 34^\circ\ 18'\ 46'' \\ \hline \end{array}$$

$$\Delta = 26^\circ\ 55'\ 14''$$

$$\Delta/2 = 13^\circ\ 27'\ 37''$$

$$R = 1909.86 \text{ (From tables or calculated.)}$$

$$L = \Delta/D = \frac{26.92056}{3}\ 100 = 897.452$$

$$T = R \tan \Delta/2 = 1909.86 \times .2393457 = 457.117$$

$$L.C. = 2R \sin \Delta/2 = 2 \times 1909.86 \times .2327712 = 889.121$$

	N	E	
P.I.	236,771.389	204,780.262	
T cos θ	- 219.985	- 400.703	T sin θ
P.C.	236,551.404	204,379.559	
R sin θ	+ 1,674.158	- 919.108	R cos θ
Center	238,225.562	203,460.451	
P.I.	236,771.389	204,780.262	
T cos ϕ	+ 377.566	+ 257.682	T sin ϕ
P.T.	237,148.995	205,037.944	
R sin ϕ	+1,076.608	-1,577.492	R cos ϕ
Center	238,225.563	203,460.452	

These computations are self explanatory. It should be noted that the coordiate values of the center check. This method of computation is analagous to the computation of a closed traverse.

COMPUTATIONS FOR A SPIRALLED HORIZONTAL CURVE

Most modern highway design requires that all curves of $1^\circ\ 15'$ D or greater shall be spiralled on both ends. In plotting and also in field layout the known coordinates of points of the spiral and shifted curve guarantee accuracy.

Example.—Suppose that the curve just computed is to be spiralled at both ends by a 300 ft spiral.

Computations.—In order to show the computations required, Fig. 5 is not drawn to exact scale. The position of the offsetted curve is exaggerated. Properties of the spiral can be taken from any spiral tables.

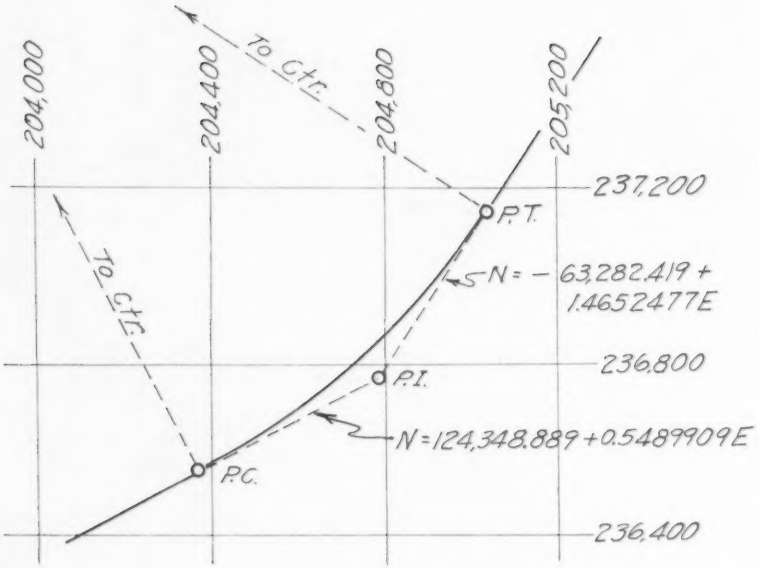
204,000
204,400
204,800
205,200
To Ctr.
To Ctr.
237,200
P.T.
S N = − 63,282.419 +
1.4652477E
236,800
P.I.
N = 124,348.889 + 0.5489909E
P.C.
236,400

FIG. 4

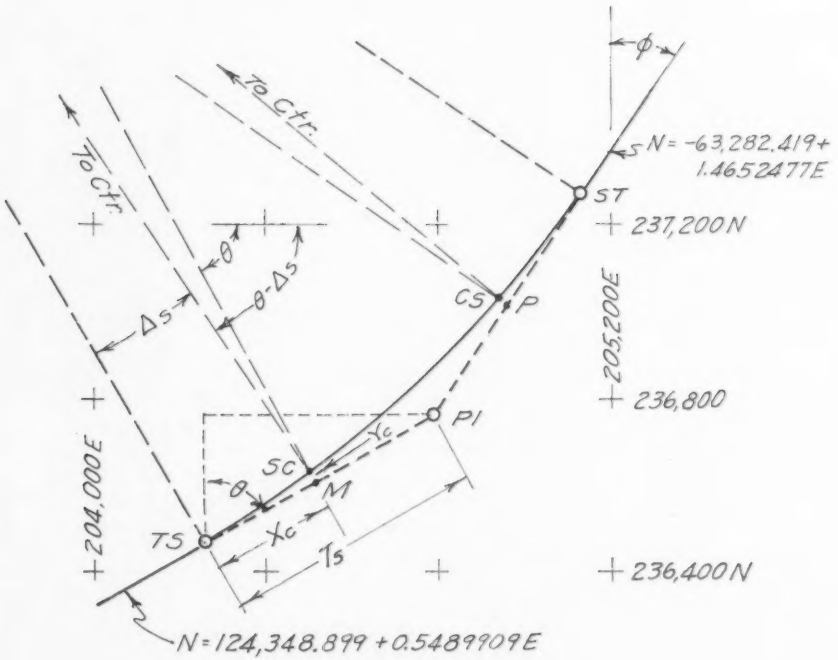
To Ctr.
To Ctr.
ϕ
S N = -63,282.419+
1.4652477E
ST
237,200N
205,200E
236,800
CS
P
PI
Δs
θ
θ-Δs
SC
M
θ
TS
Xc
Ts
204,000E
236,400N
N = 124,348.899 + 0.5489909E

FIG. 5

Spiral Angle, $\Delta s = 4^\circ 30'$ Yc = 7.85 Xc = 299.81

	N	X	
P.I.	236,771.389	204,780.262	
Ts cos θ	- 292.383	- 532.576	Ts sin θ
T.S.	236,479.006	204,247.686	
Xc cos θ	+ 144.282	+ 262.809	Xc sin θ
M	236,623.288	204,510.495	
Yc sin θ	+ 6.881	- 3.778	Yc cos θ
S.C.	236,630.169	204,506.717	
R sin (θ - Δs)	+ 1,596.885	1,047.628	R cos (θ - Δs)
Center	238,227.054	203,459.089	
P.I.	236,771.389	204,780.262	
Tx cos ϕ	+ 501.825	+ 342.486	Ts sin ϕ
S.T.	237,273.214	205,122.748	
Xc cos ϕ	- 247.635	- 169.006	Xc sin ϕ
P	237,025.579	204,953.742	
Yc sin ϕ	+ 4.425	- 6.402	Yc cos ϕ
C.S.	237,030.004	204,947.240	
R sin (ϕ - Δs)	+ 1,197.057	- 1,488.150	R cos (ϕ - Δs)
Center	238,227.061	203,459.090	

Worthy of note is that the discrepancy of the centers as computed from the C.S. and the S.C. is only 0.01. From a practical standpoint, this is a negligible error.

COMPUTING EQUATIONS FOR PARALLEL LINES

Parallel lines are used continually in all kinds of civil engineering work. The computation of equations for parallel lines is very simple but worthy of explanation.

Suppose, for example, that the line AB shown in Fig. 6 is to be shifted to a parallel position a distance of 48 ft. The bearing angle remaining the same,

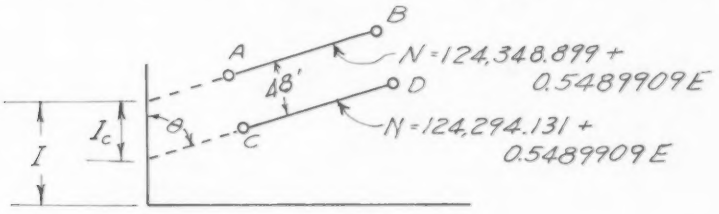

FIG. 6

the only part of the equation that changes is the intercept, I. The change in I, designated by Ic, will be $48/\sin\theta = 54.748$. The intercept for line CD will be 124,348.889 = 54.748 = 124,294.131. The equation for line CD then becomes

$$N = 124,294.131 + 0.5489909$$

PLOTTING CIRCULAR ARCS

The plotting of circular arcs to a large scale, when the center cannot be located, sometimes poses quite a problem. A common method used to accomplish this is to compute tangent offsets and plot through these with templates. Using the coordinate system it is a simple matter to compute the intersections of arc with selected grid lines.

The principle involved is shown in the Fig. 7. Applying the properties of a circle, $K = R^2 - h^2$ in which h and K is the difference between the N and E coordinates, respectively, of the center and any point A on the arc.

Let it be required to intersect the 3° curve shown in Fig. 3 with the N 236,600 grid. The solution is as follows:

	N
Center	238,225.562
Grid	236,600.000
	1,625.562

then $K = (1909.86)^2 - (1625.562)^2$. Solving, K = 1,002.556

	E
Center	203,460.451
K	+ 1,002.556
A	204,463.007

The coordinates of A are: N 236,600, E 204,463.0 to the nearest tenth, which is closer than plotting accuracy.

The computations involved in this operation may be done in less than 2 min by the average computer with an electric calculator. In this manner any number of points are computed so that ordinary curve templates can be used for drawing the arc with negligible error.

INTERCEPT ON E - W ORIGIN

When a specific job has lines running nearly parallel with the meridian it would be expedient to use equations based on the extended line to intercept the E-W origin of coordinates because the value of I approaches infinity. As shown in Fig. 8, the general equation of a line based on the intercept on the E-W origin of coordinates is $E = Ip \pm N \cot \phi$.

It frequently becomes necessary to convert an equation of a line from one system to the other. In this case the angle of the line with the E-W axis is the complement of the bearing angle and is designated as θ.

Since the tangent is the reciprocal of the co-tangent, and the equation using the intercept on the E-W origin would be the reciprocal of the equation using the intercept on the meridian.

To illustrate this principle, suppose it is required to convert the equation $N = -63{,}282.419 + 1.4652477\ E$ to the form $E = Ip \pm N. \cot \phi$. To find the reciprocal equation, divide through by the co-tangent of the angle ϕ, thus,

$$\frac{N}{1.465277} = \frac{-63{,}282.419}{1.4652477} + \frac{1.4652477\ E}{1.4652477}$$

Solving: $0.682\ 4785\ N = -43{,}188.88 + E$

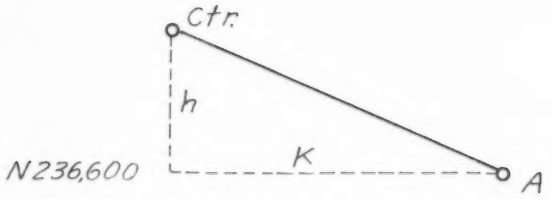

FIG. 7

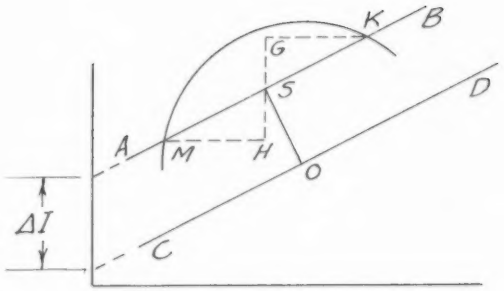

FIG. 9

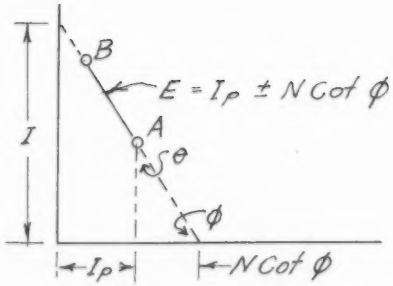

FIG. 8

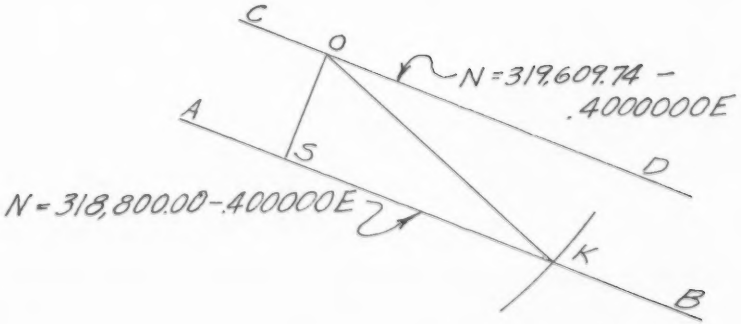

FIG. 10

Rearranging yields

$$E = 43,188.88 + .62824785\ N$$

which is the equation sought.

The proof of the accuracy of the equation is to substitute known coordinate values to determine if the points will satisfy the equation using the N value of the P I of Fig. 3 in the equation, it becomes:

$$E = 43,188.888 + .682,4785\ (236,771.389)$$

Solving this yields E = 204,780.262, which is correct.

INTERSECTION OF CURVE AND LINE

A problem frequently encountered in highway design is to find the intersection of an arc of a curve and a straight line. An example would be an overpass on a curve.

To illustrate the principles and methods involved, a simple diagram, Fig. 9, will be used. Following this procedure a hypothetical problem will be presented.

To solve the problem the known quantities are the equation of the line AB, the coordinates of the center of the curve, and the radius of the curve.

By analysis of Fig. 9, the following steps are used:

1. Compute the equation of a line, CD, through the center of the curve and parallel to line AB.
2. From equations AB and CD find ΔI.
3. The perpendicular distance OS, from the center of the circle to line AB is $\Delta I \sin \theta$.
4. From geometry, a perpendicular from the center of a circle to the chord bisects the chord. In triangle OSK, solve for SK.
5. Starting at 0 compute coordinates of S and K.
6. For a check use coordinates of K to see if they satisfy equation of line AB.

Example.—Given a line AB whose equation is N = 318,800 = .4000000 E, and which is intersected by a 3° curve whose center coordinates are N = 238,225.56, and E = 203,460.45. It is required to find the coordinates of the point of intersection. The solution is as follows, with reference to Fig. 10.

Step 1.—Equation of a line through the center of the curve and parallel to line AB is

$$N = I - .4000000\ E \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (9)$$

Substituting values of the center of the curve the equation becomes

$$238,225.56 = I - .4000000\ (203,460.45)$$

Solving this gives

$$I = 319,609.74 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (10)$$

Step 2.

$$\Delta I = 319,609.74 - 318,800 = 809.74$$

Step 3.

$$OS = \Delta I \sin \theta$$
$$= (809.74)\ (928\ \ 4758)$$
$$= 751.825$$

Step 4.

$$SK^2 = R^2\ OS^2$$
$$SK = \sqrt{1906.86^2 - 751.824^2}$$
$$SK = 1752.309$$

Step 5.

	N	E	
Center	238,225.56	203,460.45	
OS sin θ	- 698.05	- 279.22	OS cos θ
S	237,527.51	203,181.23	
SK cos θ	- 650.79	1,626.98	SK sin θ
K	236,875.72	204,808.21	

Step 6.—Substitute values of K in equation of line AB.

$$236,876.72 = 318,800 - .4000000\ (204,808.21)$$

Solving this yields

$$236,876.72 = 236,876.72$$

Proving that the values of K satisfy the equation of line AB and is therefore correct.

CONCLUSION

The method of line equations and the checks involved guarantee against errors. Furthermore, no independent check by another computer is needed. Highway plans can be simplified by labeling center-line or edge-of-pavement lines with an equation which is all the information needed for field layout. By the use of the coordinate system any ground points that have been destroyed may be replaced from other known points.

ACKNOWLEDGMENTS

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Journal of the
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SURVEYS AT COUGAR RESERVOIR[a]

By Charles N. Oros[1]

SYNOPSIS

An outline is given of surveys made in the Willamette Valley preliminary to the special surveys at Cougar Reservoir and Dam Site. Factors which must be considered in large scale photogrammetric mapping of precipitous high relief areas are presented and how these problems were resolved using projector type stereoplotting equipment.

This paper presents a chronological story of the surveys at Cougar Reservoir located about 50 miles east of Eugene, Oreg., on the west slope of the Cascade Range. It emphasizes the part photogrammetry contributed to the project. Surveying is mentally pictured as something accomplished in the field, out in the open, using tape, transit, level and axe; it is, although to a much lesser degree, for a photogrammetric survey. Without the basic data gathered in the field and transferred to the photographs as picture-point control, an accurate photogrammetric map cannot be made.

During the years 1935 and 1936 extensive surveys were made in the Willamette River Basin. Many monuments were set, over which traverse and level lines were run. Aerial photographs were taken of all the cultivated areas and reservoir sites. Plane table surveys and geology investigations were made of proposed dam sites and reservoirs. From this data reports were written and plans formulated to build dams that would control the recurring winter and

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a Presented at the June 1958 ASCE Convention in Portland, Oreg.

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spring floods, improve navigation and irrigation through regulation of stream flow, and develop electric power. Since the initial report there have been further site investigations and reports, design and construction surveys, as well as actual construction of dams.

Wherever possible the surveys were tied and adjusted to United States Coast and Geodetic Survey triangulation and levels. Without exception this was possible for the level lines, but the traverse lines fared poorly. Only those in the main valley of the Willamette River were tied and computed on either geodetic or Oregon State coordinates. The reservoirs were in areas that would have required considerable time and expense to have tied them into the then existing triangulation network. However, during later years, the United States Coast and Geodetic Survey, the United States Forest Service, the United States Geological Survey and the United States Army, Corps of Engineers, have extended the triangulation network so that all of the constructed dams and most of the sites now under investigation are tied into a common datum - the Oregon State Coordinate System.

To date, all surveys, plans, and designs at the Cougar Reservoir site have been worked on a local coordinate system. During 1957 a United States Geological Survey triangulation party made a geodetic tie to the main traverse while establishing control for their quadrangle mapping. Consequently, in time, all of the Cougar Reservoir data will be converted to Oregon State Coordinates.

In 1945 the reservoir area was in virgin timber along the South Fork of the McKenzie River, accessible only by a Forest Service Road adjacent to the river. During that year a triangulation monument was placed on each abutment as reference points for future surveys. They were tied into a short traverse run at that time, which was computed on local coordinates. Later, reconstruction of the road destroyed all but one of the traverse points.

During 1953, the Cougar Reservoir became an active project. Two-in. iron pipes with railroad spikes cemented therein were set along the Forest Service Road throughout the length of the proposed reservoir. From the triangulation monuments on each abutment a traverse was run through the new monuments and computed on the original local coordinate system. Downstream about 3 miles from the dam site there is a United States Coast and Geodetic Survey first order level line along the McKenzie River Highway. From this line a single run level line was run to establish elevations on the monuments set along the road. These monuments, on 1929 datum and local coordinates, became the basic control for all subsequent surveys for the planning, geology, design, relocation, cadastral and construction phases of the program.

The first bit of topographic mapping was the dam site area. Several plane table sheets at a scale of 100 ft per in. with 2 ft and 5 ft contour intervals were completed during the summer of 1953. At the same time aerial mapping photography was taken of the reservoir area with an $8\frac{1}{4}$ -in. camera. Picture points were selected and described on a set of photographs which were then sent to the field for a contract survey crew to make proper ties. Horizontal picture point control was chosen along the Forest Service Road, with the points as close as possible to the traverse-level line monuments. Finding suitable places for vertical picture-point control was very difficult because the area was covered with timber up to 160 ft or more in height. Occasionally a small open area would appear in the timber or a stream junction could be determined on a photograph. Elevations were obtained at these points as well as along the Forest Service road that meandered somewhat through the center of the flight strip.

In open or scattered brush country, 5-ft contour interval 200 ft per in. maps

Photogrammetrically speaking, the multiplex extensions made from the foregoing control left much to be desired because there were not enough vertical picture points per model. Also, absolute readings could not be taken in many cases, and pass points in tall, dense timber are uncertain. These tend to leave the stereoscopic instrument operator with a feeling of uncertainty. Nevertheless a topographic map of the reservoir area was completed in 1955 at a scale of 1:9600 with a 20-ft contour interval.

The question may be raised: How can a contour be delineated correct to plus or minus 10 ft when it is not possible to see the ground because of timber 160 ft or more in height? The answer is that it cannot be done. Neither can it be done by any field method at anywhere near a comparable cost. To obtain the 8 horizontal and 45 vertical picture points requested to prepare said reservoir map of 14 sq miles, the field costs were more than the total photogrammetric costs, including contract payments for flying, making mosaics, drafting of dam site topography and other incidentals.

In spite of the unfavorable conditions for preparing this topographic map it is surprising how good it actually is. During 1956, preliminary lines for road relocation were run on both sides of the proposed reservoir at about elevation 1,700 ft mean sea level, just above the full pool elevation. Every seventh station for approximately 8 miles along the right bank and 2-1/2 miles along the left bank was plotted, without favoritism. In fact, in order to check the stereoplotting work, the right bank section was chosen to go through the area of greatest uncertainty. Of the 60 points checked without benefit of horizontal movement as is provided in map accuracy specifications, 36, or 60%, were correct within one contour interval — that is plus or minus 20 ft. Similarly, 15, or 25% were plus or minus 20 ft to 50 ft in error and the remaining 9, or 15%, ranged up to 150 ft in error. The maximum horizontal displacement of any contour at a point checked was 0.15 in. at the map scale of 800 ft per in., with only 10, or 17%, displaced over 0.10 in. and 25, or 40%, under 1/30 in.

The results cited are normal at this office. At Hills Creek Reservoir, on the Middle Fork of the Willamette River, with similar terrain, ground cover, control, mapping and checking, the accuracy results were almost identical for a 1:12,000 scale map. Also, a multiplex set-up indicated an error of approximately 100 ft at a certain point in the basic traverse. A field check verified that one chain length had been dropped in the original work.

At this point a word of caution, or perhaps advice or suggestion is appropriate. It is believed that the preceding results of the photogrammetric surveys are very satisfactory. However, it is not possible to purchase a stereoplotting instrument, pick up a man to operate this type of precision equipment, and then expect to obtain similar satisfactory results. Each person concerned with these phases of photogrammetric mapping had fifteen or more years of experience in his specialty. This experience, combined with concientiousness, is essential. So, also, is the right kind of aerial photography and picture point control. Even then, absolute accuracy is not obtainable under the foregoing conditions. Nor is it obtainable by ground surveys, unless a tremendous amount of money is expended.

The dam site and reservoir maps described were used for reports, planning, and design until larger scale maps of greater accuracy were required. For road relocation, cross sections were taken from the before mentioned preliminary lines. The usual process of plotting notes, projecting an alignment, computing cuts and fills to establish a line and staking the located line for construction sufficed for most of the road relocation.

In open or scattered brush country, 5-ft contour interval 200 ft per in. maps are stereoplotted. These maps are used to project the proposed road and/or railroad relocation alignment, thereby eliminating the field work of running a preliminary line and taking cross-sections. That reduces overall costs considerably, because only the located line needs to be staked.

By 1956 the Cougar Dam Site area had been cleared. As design progressed, serious discrepancies appeared in the 100 ft per in. plane table topography. This map, made before any clearing of the site, was a compromise between accuracy, cost, and safety. Not only were insufficient rod readings taken because of the dense brush and timber, but much of the area is inaccessible. Consequently, during the latter part of 1956, a major portion of the dam site was mapped photogrammetrically to replace the original survey.

In the spring of 1957 stripping of overburden was started. This operation was not necessary in the rock outcrop area of near vertical and overhanging cliffs at each abutment, but excavation quantities for the spillway, road relocation and main dam would have to be determined and paid for. The thought of accidents to surveymen hanging by ropes in order to take cross sections created the following questions "Could it be done photogrammetrically, at 40 ft to the in. with elevations correct to 1 ft?"

Here was a problem, a challenge to photogrammetry, that was accepted with reservations and solved with compromises and unorthodox procedures. Some of the problems and their solution are given in the following paragraphs.

The river elevation is approximately 1,280 ft and the clearing line above the abutments around 2,200 ft, mean sea level, a difference of 900 ft. At 40 ft per in. the vertical movement of the tracing table must be 22.5 in. Optimum projection distance for the plotting equipment is 21 in. plus or minus 4 in. This can be stretched to plus or minus 5 in. which would allow a 10 in. vertical throw, but 22.5 is needed.

A map scale of 40 ft per in. required a negative scale of 136 ft per in., or 1:1630. Flight altitude above mean datum using a 6-in. lens would be 815 ft. Mean datum of 1,750 plus 815 is 2,565 ft mean sea level or only 365 ft above the cleared areas at the timber cutting line. Maximum forward lap that can be accommodated in the stereoplotting equipment is 71%. Net forward movement of the airplane for 71% overlap on a 9-in. negative at a scale of 136 ft per in. is 355 ft. It is then determined that 355 ft is 50% overlap for a negative scale of 79 ft per in. Flight altitude for that scale is 475 ft above ground. Therefore elevation 2,090 ft mean sea level, 2,565 minus 475, would be the upper limit of mapping photography coverage. That is not high enough. Also, to obtain exposures 355 ft apart, they would have to be taken at 4-sec intervals from a plane flying at 60 mph. Normally, precision aerial mapping cameras do not operate that fast and 60 mph is not a safe flying speed when the timbered hills are just off the wing tips.

From the foregoing it can be seen that every phase of the operation would be right at or beyond the limit of equipment capability, with no tolerances available. Obviously another approach to the problem should be taken.

Just twenty years ago it was considered that the solution just could not be done. However, in a slightly different approach, the author's office has been doing it occasionally for the past fifteen years. The solution was to take the photography with an 8-1/4-in. camera and use it in a 6-in. projector.

The difference in elevation in a single model is approximately 900 ft. With a vertical plotting scale of 70 ft per in., the vertical range of the floating mark

must be 900 divided by 70 or 13 in. This is stretching the 10-in. range of sharp focus, but it remains satisfactorily workable. With matched equipment, that is 6-in. photography used in a 6-in. projector, the horizontal and vertical scales are identical. But using unmatched equipment, that is 8-1/4-in. photography in a 6-in. projector, the vertical scale is 69.44 ft per in. and the horizontal scale is 50-ft per in.

So, in addition to 8-1/4 in. photography, the horizontal map scale of 50 ft per in. was accepted. Photographic scale for a 50-ft per in. map is 170 ft per in. or 1:2040 and would require a flight altitude above datum of 1,400 ft instead of 815 ft. That allows some tolerances in the several factors previously considered. Further, although it was necessary to use the lowest areas for control in order to set up the models, it was not necessary to contour below elevation 1,500 ft mean sea level. Consequently, the mean datum was taken at 1,800 ft. Also, it was decided to work the stereoplotting equipment on the long side of the optimum projection distance. Combining all these factors would allow a little more flight elevation above ground.

Other complications were caused by the east-west direction of the axis of the dam. First, the steepest part of the abutments is on the north side, which puts that side in shadow. Second, crevices into each abutment accentuated the problem. By observation, the project engineer determined minimum shadow at the west abutment occurred at 10:15 ante meridian and for the east abutment at 2:45 post meridian. A contract to fly at those hours was consummated. Other specifications were: Flight altitude, 3,300 ft mean sea level; exposures pinpointed; two flight lines over each abutment; two passes over each line with both a 6-in. and an 8-1/4-in. camera; camera and lens passed by the United States Bureau of Standards for aerial mapping photography; and all possible precautions taken to obtain truly vertical exposures at the specified elevation.

Pinpointed exposures were requested in order to have the exposure stations at points where minimum blind spots would occur. In other words, the best perspective points for stereoscopic vision of the sheer rock walls, crevices, and overhanging cliffs were necessary.

Six-in. photography, duplicating the 8-1/4-in., was taken to insure success and for checking. Under the conditions being considered, that is, 8-1/4-in. photography being used in six-in. projectors, the aerial photography must be perfect. That means, no tip or variation in flight altitude can be tolerated. For that reason, and to further insure success, two runs over each strip were requested. Air currents in the canyon, slow speed of the plane, likelihood of incomplete vacuum, lack of overlap and missing the exposure stations were too many hazards to expect success with just one run. By the time the actual setting up and working of the models could take place, re-flights would be out of the question because the sun altitude would be too low.

Prior to the date for flying, crosses, made from two by eight-foot pieces of plyboard painted white, were placed over all the monuments in the area. There were 13 in all, tied together by triangulation and levels. After receipt of file prints, two additional horizontal and several vertical control picture-points were marked on a set of photographs. These were sent to the field for appropriate ties. Most of the vertical picture point elevations were tops of stumps, plus the elevation of the ground on the uphill side of the stump.

Except for three small areas in complete shadow or with overhang, a topographic map was made of the entire dam site construction area and portions of the road and quarry areas. A 2-ft interval was used throughout, but the horizontal scale varied from 40 to 69.44 ft per in.

The right abutment manuscript was prepared with one exposure from each flight line of the 8-1/4-in. photography. This permitted the best stereoscopic vision into the crevices. To obtain the north face of that rock outcrop, two 6-in. photographs were selected, one from each flight line. Map scales for the manuscripts were 40 and 60 ft per in., respectively. Because contours lower than 1,700 ft were not obtained in either of the above set-ups, a third pair, using 6-in. photography, was set up at a scale of 69.44 ft per in. Approximately one-half of the area contoured was duplicated in each of these three set-ups. This afforded an opportunity to cross check the work. For identical points, differences in elevation were less than a foot; for position, it was plotting ability. To compare contours drawn in the overlap area is difficult to explain, or to arrive at definite conclusions. In clearly defined areas, areas of no shadow or blind spots, a contour displacement of 0.04 to 0.02 of an in. on the map, corresponding to less than 3 ft on the ground, did happen. Generally, they crossed with maximum deviations of 0.02 in., approximately 1 ft on the ground. But even this slight drift may mean 80 ft difference in elevation, instead of 1 ft which would be one-half of the contour interval, generally considered as a criterion in accuracy specifications.

Most of the left abutment was mapped at 50 ft per in. using 8-1/4-in. photography. Due to slight variation in flight altitude of the 8-1/4-in. photography, the quarry area to the south and road relocation area to the north had to be worked with 6-in. photography at 62.5 ft per in. Here again there were overlapping areas which provided a check on the stereoplotting work: different models, different or additional control, different cameras, and different flight strips. Results of these checks were the same as those for the right abutment. Also, the P_2-line for the west side road relocation was plotted after completion of the map. Maximum discrepancy of the six points which fell in the contoured area was 1 ft.

The various manuscripts described were brought to a 50-ft per in. scale for tracing. This set is the final or basic topographic map for the dam site. From it, excavation quantities will be computed for contract payments. It also was used for projecting the road through the left abutment area. Reduced to 100 ft per in., a positive transparency was prepared with only the 50-ft contours retained above the 1,300-ft elevation. This became the basic topographic dam site map on which appropriate data were superimposed for each of several contractual features.

Many models were tried that came very close to setting up but were not acceptable because of a small amount of parallax someplace in the model; an elevation difference of 1/2 ft or more at a vertical picture-point control station; or a one-foot or more displacement at a triangulation point. Hours, even days, were spent bringing the models used into absolute orientation, and parallax free. In addition, the area of each model used was carefully correlated with the area of least distortion for the lens of the taking camera as determined by the United States Bureau of Standards test for that lens. Whenever these precautions were overlooked, horizontal displacement of contours became excessive—in a few instances as much as a tenth of an in. at the map scale.

In conclusion, the foregoing narration has dealt with the major surveys of the Cougar Reservoir site on the South Fork of the McKenzie River east of Eugene, Oreg. There were other surveys at the site, construction surveys will continue throughout the construction period, and there will be various surveys after construction is completed.

Journal of the
SURVEYING AND MAPPING DIVISION
Proceedings of the American Society of Civil Engineers

SDR SURVEY FOR PROPOSED CHESAPEAKE BAY CROSSING[a]

By Walter C. Beckmann,[1] Charles L. Drake,[2] and George H. Sutton[3]

SYNOPSIS

The application of a wide band, spark source echo sounder, the Sub-bottom Depth Recorder (SDR) to foundation studies for the proposed Chesapeake Bay Highway Crossing from Little Creek to Cape Charles, Virginia, is described in detail. Reflection horizons are correlated with varying properties of the sediments as revealed in test borings.

INTRODUCTION

Only in recent years have geophysical techniques been used to aid in the solution of problems encountered in foundation engineering. This is due to the novelty of the techniques, to the concentration of geophysical efforts in other fields, and to the unfamiliarity of the foundation engineer with geophysical methods. The application of many of the tools developed for geophysics can offer not only a saving of time and effort to the foundation engineer but furnish him with additional information not made available by standard practices.

An excellent example of this is a recently developed sonic reflection equipment, the Sub-bottom Depth Recorder (SDR). This equipment is similar to

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standard echo sounding equipment used to determine water depth. Utilizing high powered, low frequency sound sources and variable pass ban detection, it permits the mapping of not only the bottom topography but the structure beneath the bottom as well.

Using the SDR, it has been possible to map sub-bottom horizons at depths exceeding 1,600 ft in regions with high acoustic impedance contrasts. Of equal importance is the fact that by judicious selection of pass band frequencies very small changes in the lithology of the sedimentary column may often be distinguished and clearly mapped.

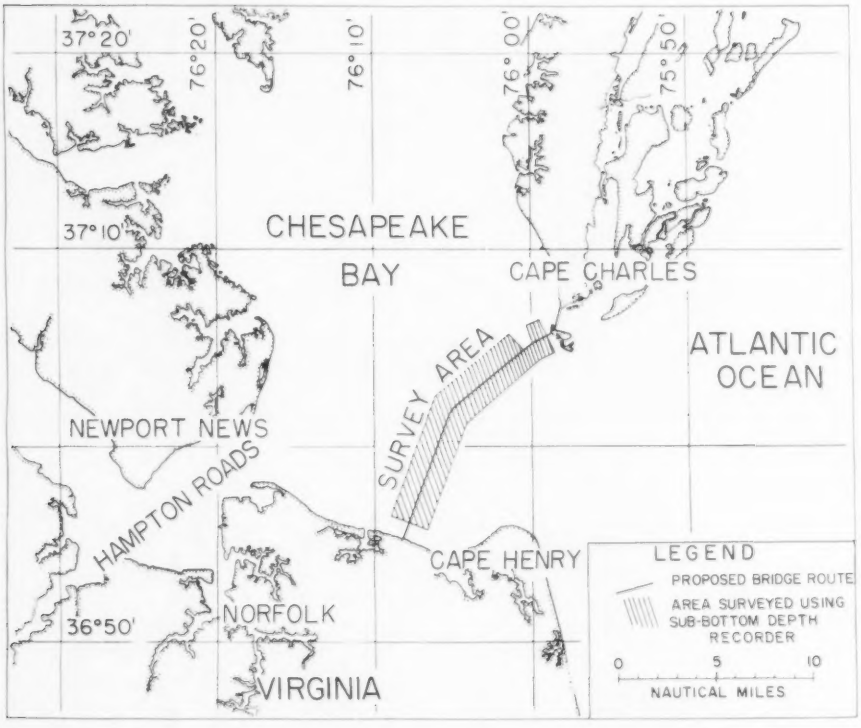

FIG. 1.—LOCATION OF SURVEY AREA

During the latter part of January 1958, a SDR was used to survey the area of the proposed Chesapeake Bay Highway Crossing between Cape Charles and Little Creek, Virginia, shown in Fig. 1. The survey area (shaded) parallels the highway crossing route extending a mile on either side. The area was in open water and subject to weather and seas entering Chesapeake Bay from the Atlantic Ocean. In this shaded area a one half mile grid was surveyed covering over 100 miles of track. Lines of particular interest were surveyed several times using various filter and gain parameters. The grid lines are shown subsequently in Fig. 4. Although there were no extensive interfaces with high

impedance contrasts in the survey area, it was possible to map a number of horizons within the relatively homogeneous sediments.

DESCRIPTION OF EQUIPMENT

The SDR is similar to a sonar echo sounder. The model[4] used in this survey utilizes a broad band, low frequency sound produced in the water by an electrical spark discharge. The energy reflected from the bottom and sub-bottom horizons is detected by a towed pressure-sensitive hydrophone. The signal is suitably amplified, electronically filtered, and displayed on a chart recorder. Sample records are shown in Fig. 2(a) and (b).

The record shown in Fig. 2(a) was made on Long Island Sound using the Sub-bottom Depth Recorder (SDR). The outgoing signal corresponding to the water surface is printed at the top of the record. The first reflection corresponds to the water bottom)composed here of recent sediments) while the various sub-bottom reflections correspond in this instance to the top of the glacial deposits and to the top of the bedrock. At the extreme right of this record the edge of the coastal plain sediments, resting unconformably on the bedrock, can be seen. Receiver pass band 250 to 750 cps.

The record shown in Fig. 2(b) was made using the Sub-bottom Depth Recorder (SDR) in Chesapeake Bay along the route of the proposed highway crossing near bore hole D2. The sub-bottom reflections are from horizons within the Miocene and post-Miocene sediments underlying the survey area. These sediments are flat lying for the most part.

The sedimentary column does not undergo any great changes such as shown in Fig. 2(a). However, by proper selection of equipment parameters readable and useful records were obtained. Receiver pass band 400 to 4,000 cps.

The arrivals on this record (Fig. 2(b)) corresponding to reflections from the various horizons have been emphasized by pencil lines. Light portions of record are from reduced gain to obtain sharper observation of water and shallow sediment depths.

The electrical spark discharge which produces the necessary accustic energy is achieved by discharging a charged capacitor across a small spark gap, which is towed in the water behind the survey vessel by a small electrical cable. The resulting sound is a high level, broad band pulse of short duration. The paramount advantage of a broad band sound source is that sediments will selectively reflect various frequencies of sound energy depending on the physical characteristics of the sediments.

The ability, through the use of electronical filtering, to select the optimum frequency band of reflected signal permits one to adapt the SDR to different sub-bottom conditions. The higher frequencies were used in this area to produce a cleaner, more detailed record since the physical changes in the sediments were small and in many cases the thicknesses involved were slight.

The ratio of incident to reflected energy for a given horizon depends chiefly on the acoustic impedance change across the interface and on the ratio of the wave length of the sound used to the dimensions involved. In general the lower frequencies penetrate deeper into unconsolidated sediments and the higher frequencies are preferable for studies of fine structure. In an area such as Long

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[4] "Sub-Bottom Depth Recorder," by Walter C. Beckmann, Archie C. Roberts, and Bernard Luskin, Geophysics, Vol. 24, No. 4, October, 1959, pp. 749-760.

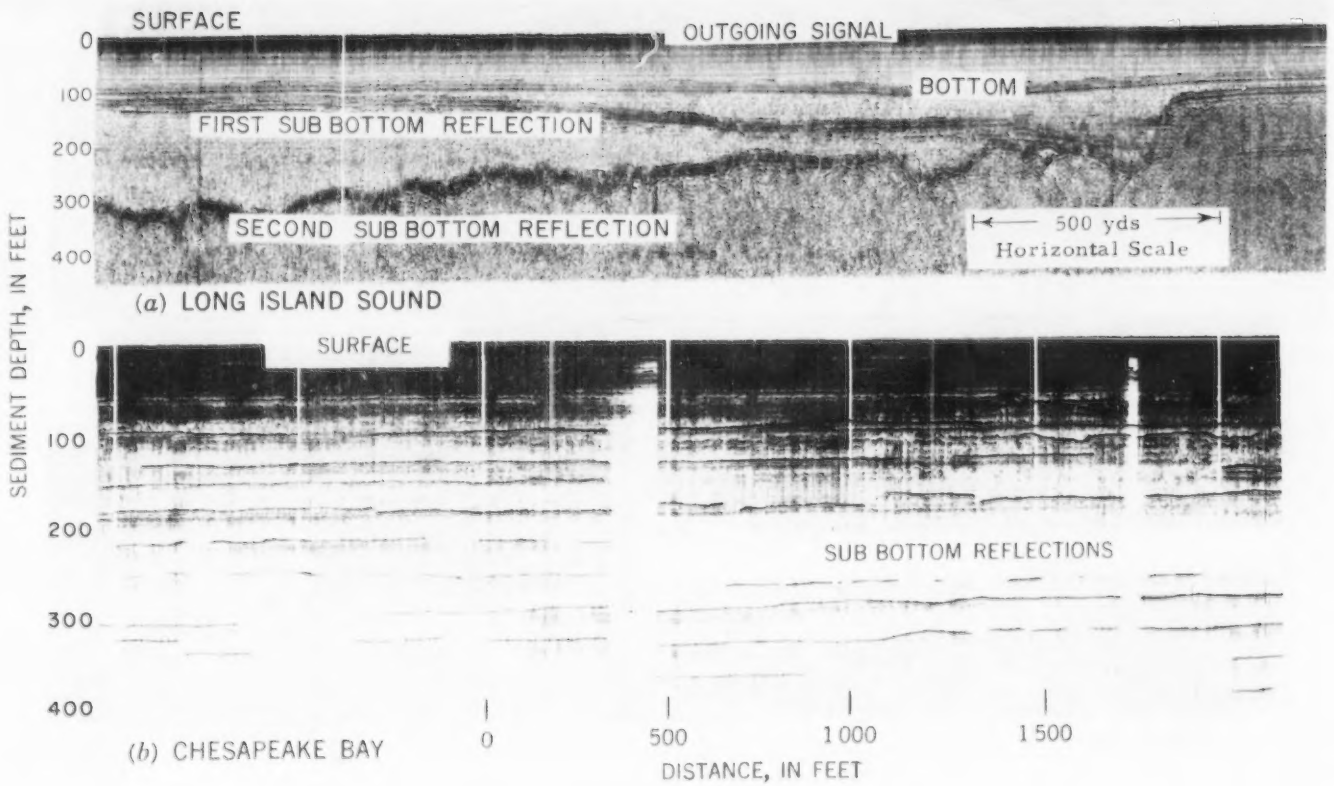

FIG. 2.—TYPICAL SDR REFLECTION RECORDS

Island Sound where the sub-bottom horizons have large impedance contrasts and where the thicknesses of these horizons are large, use of high frequencies is not advantageous, and one can achieve a better signal to noise ratio by utilizing lower frequencies. In fact the highest quality records from this area are made using a pass band of 250-750 cps (Fig. 2(a)). In the survey area the records made were of a less contrasting nature since this is the case with the sediments themselves. The records however were readable and provided the required data (Fig. 2(b)). One of the principal advantages, then, of a broad band sonar such as the SDR is the fact that the pass band may be selected as sub-bottom conditions change, even within a particular survey area.

The SDR is contained in two standard 19 in. relay racks 40 in. high, housing the transmitting and receiving sections. A small "fish" is towed about 150 ft behind the survey vessel; this contains the receiving hydrophones and trails the spark source.

The equipment is easily installed aboard a survey vessel in less than an hour and requires no attachments or modifications to the ship's hull. The power requirements are 2 kw of 115 vac from an unregulated source. Precision 60 cycle power for accurate timing is controlled by a tuning fork oscillator.

The survey vessel used for the survey was the 70 ft yacht, "Robin," fitted with a small gasoline generator mounted on deck. Geographic position of the vessel was determined by "Raydist."[5,6] A continuous plot of Raydist coordinates was made on a two channel Brush Recorder. In addition, positions were read and plotted each minute with the result that excellent position control was possible. A reasonable estimate of error of position is ± 30 ft. A portion of the detailed navigation plot is shown in Fig. 3 in the region of the Baltimore Channel. The small circles correspond to positions determined by the Raydist navigation system at 1 min intervals. Raydist was used to produce a continuous plot of the track of the survey vessel and was used to control the vessel. The coordinate grid shown has a 10,000 ft interval and refers to Virginia State Coordinate System (South Zone).[7]

INTERPRETATION OF DATA

Determination of Velocity of Sound.—The parameter which is measured in echo sounding or seismic reflection techniques is reflection time. In order to relate the measured times to depth, it is necessary to know the velocity of sound in the media which are penetrated. The sound velocity in water may be easily determined from hydrographic data but sound velocities in the sediments require additional knowledge.

The Chesapeake Bay Institute of Johns Hopkins University has made numerous hydrographic measurements in Chesapeake Bay. Among their regularly occupied stations are five that are near the proposed bridge line. For the period January 10-21, 1951, a mean temperature of 5 C and a mean salinity of

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[5] "Raydist Locates Boring Sites for Chesapeake Crossing," by Allen L. Comstock and P. Z. Michener, Civil Engineering, July, 1959, pp. 54-57.

[6] "Transistorized Raydist as Used in Geological Surveys," by Charles L. Drake and Walter C. Beckmann, Journal of Geophysical Research, Vol. 65, No. 2, February, 1960, pp. 525-528.

[7] U. S. C. and G. S., U. S. Dept. of Commerce, Special Publications 325 and 329.

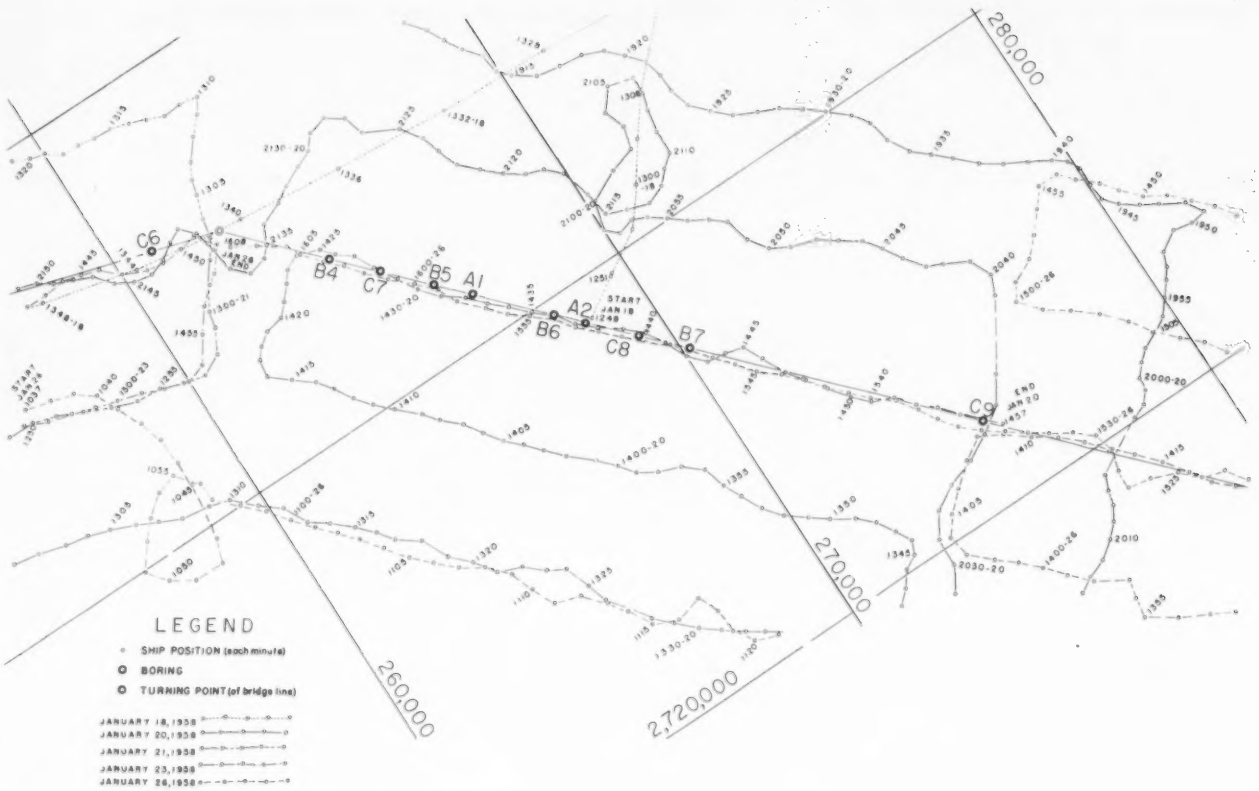

FIG. 3.—DETAILED NAVIGATION

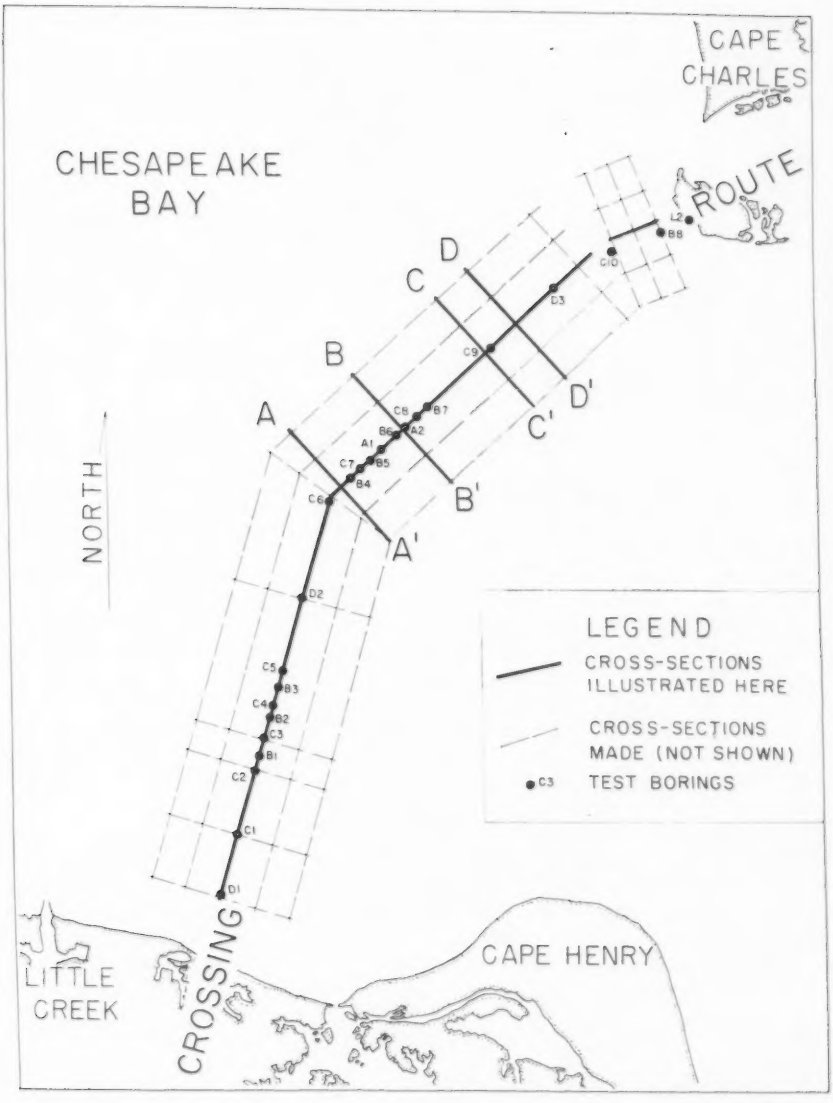

FIG. 4.—SECTION PLAN

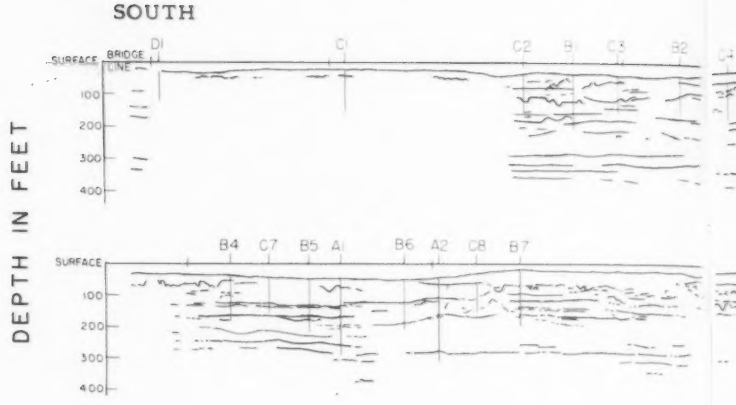

FIG. 5.—CROSS SECTION ALONG

28°/00 were determined.[8] The resultant computed sound velocity[9] is 4,800 ft per sec.

The variation of velocity with depth in the coastal plain sediments has been studied by J. E. Nafe and C. L. Drake[10] and a mean gradient has been determined. Using this gradient, a mean velocity of 1.2 times the velocity of sound in water was determined for the upper 400 ft of the sedimentary column. This value was used for the preliminary reading of the records.

To determine the value of sound velocity to be used for the final interpretation, comparison was made with the data. The relation between seismic compressional wave velocity and other properties (porosity, grain size, sorting, mineral content) has been studies by G. H. Sutton, H. Berckhemer and Nafe.[11] This work was carried to higher velocity values by Nafe and Drake.[10] For the type of sediments encountered in the area of the proposed highway crossing and for the porosity range indicated, porosity is the major factor contributing to velocity changes.

Velocities were computed using the velocity-porosity relationship of Nafe and Drake,[10] from porosity data of samples taken from the test borings furnished by Moran, Proctor, Mueser, and Rutledge.[12] The results show a mean value closer to 1.3 times the velocity of sound in water. This difference is due partly to the low velocity of sound in the cold water, partly to unloading of the sedimentary column by erosion, and partly to selective sampling. The first factor will cause an increase in the velocity difference if, due to poor thermal

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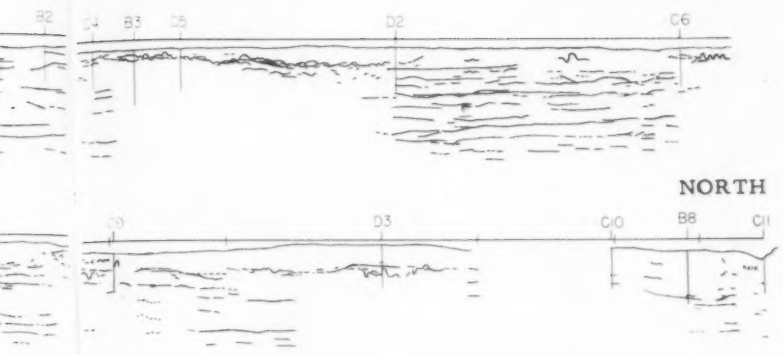
8 Chesapeake Bay Inst., The Johns Hopkins Univ., Data report, Cruise VIII, Jan. 10 to Jan. 23, 1951, Report No. 6.

9 "Tables of the Velocity of Sound in Pure Water and Sea Water for use in Echo-Ranging and Sound Ranging," by D. F. Matthews, Publication H. D. 282, Hydrographic Dept., Admiralty, London, Second Edition, 1939.

10 "Variation with Depth in Shallow and Deep Water Marine Sediments of Porosity, Density and the Velocities of Compressional and Shear Waves," by J. E. Nafe and C. L. Drake, Geophysics, Vol. 22, July, 1957, pp. 523-552.

11 "Physical Analysis of Deep Sea Sediments," by G. H. Sutton, H. Berckhemer, and J. E. Nafe, Geophysics, Vol. 22, October, 1957, pp. 779-812.

12 "Chesapeake Bay Highway Crossing," by Moran, Proctor, Mueser, and Rutledge (Sverdrup and Parcel, Cons. Engrs.), Foundation Study, Unpublished report, 1958.



PROPOSED HIGHWAY CROSSING ROUTE.

conductivity, the water is appreciably colder than the underlying sediment.[13] The second factor will be significant if the sediments were once buried to greater depths and have been exposed subsequently by erosion. In these circumstances, the velocity tends to decrease more rapidly than the porosity increases. Lastly, the water content was not measured on those high porosity samples from which it settled out. Inclusion of these would have lowered the mean velocity computed from the porosity measurements. Because of these factors the value 1.2 times velocity in water used in the preliminary readings, was also used for the final interpretation.

In addition to the velocity-porosity comparison, the computed reflection depths were compared with available boring information. The prominent reflecting horizons correlated with changes in lithology recorded on the drilling logs, confirming the choice of sediment velocity.

Accuracy of Depth Determination.—The scale of the SDR records is such that 1 ft of water depth is represented by 0.0144 in. and 1 ft of sediment depth by 0.0120 in. The reading error, as such, can probably be held to about 1 ft. Other errors arise, however, from the low frequencies used. In order to penetrate to any depth in the sediments, it is necessary to use rather low frequencies. Echo sounders, to measure water depths, may use frequencies between 12 and 200 kilocycles; standard seismic reflection prospecting techniques use frequencies ranging from 20 to 500 cycles. For work of the type called for by this survey, frequencies between these two ranges are used.

With a lowering of signal frequency there results a corresponding decrease in definition. A 200 kilocycle echo sounder uses wavelengths of 0.025 ft while a 400 cycle arrival on a sub-bottom sounder has a wavelength of 12 ft. In the latter case, the indicated depth to a reflecting horizon could be as much as 6 ft too great. The measured depths, furthermore, are somewhat sensitive to the gain setting of the receiving circuit. When higher gain is used, emergent signals are recorded at earlier times with possible reductions in apparent depth up to 6-8 ft.

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[13] "Sound Velocity vs. Temperature in Water-Saturated Sediments," by G. Shumway, Geophysics, Vol. 23, July, 1958, pp. 494-505.

Assuming that the average vertical velocity chosen was correct, and since the important sources of error tend to increase the indicated depth, a reasonable estimate of the error in the depth determination of a horizon is +10 to -2 ft. As previously indicated, the sediment velocity used has been determined to within about ± 5%. Using this velocity, the prominent reflection horizons agree with the bore hole data. Errors in the velocity used will, in general, produce errors in depth proportional to depth.

PRESENTATION OF DATA

No attempt has been made to present the multitude of SDR data which was obtained. Only examples of the type of data furnished by the SDR survey are included here. The portion of the survey in the region of the Baltimore Channel along the proposed route is considered as a particular example.

One of the principal reasons for conducting the SDR survey was to establish whether or not a buried, ancient channel of the Susquehanna River crossed the proposed bridge line. This possibility had been suggested by earlier geological studies in the area.[14] The widely spaced preliminary borings could easily have left such a channel undetected and its presence could alter considerably the foundation considerations.

Over a large part of the area surveyed many reflecting horizons were found to depths of about 350 ft. In general these were flat lying or had small dips. In certain areas the upper part of the section was complicated by minor channeling and other depositional or erosional processes at shallow depths. The most persistent horizons were at 150-160 ft and 270-310 ft. From Fig. 7 it can be seen that in the Baltimore channel area these can be correlated with layers in the boreholes which were difficult to penetrate and/or were of low porosity. The persistence of the upper of these two layers along the entire crossing line (Fig. 5) and on the sections on either side of the bridge line makes it extremely doubtful that the old channel of the Susquehanna could pass through the area that now forms the mouth of the Chesapeake Bay.

The total thickness of the Miocene in this area[15] is on the order of 600 ft. Since the deepest reflections received during the survey were from horizons about 350 ft deep, it must be concluded that the data are confined to the Miocene and the varying amounts of younger sediment above it.

Fig. 4 shows the sections made from the SDR data. The dashed lines indicate the longitudinal and transverse cross sections that were made using the SDR data, and the heavy lines indicate the cross sections which are shown in Figs. 5 and 6. The section along the crossing route is shown in Fig. 5. It represents a composite of records made at different receiver gain and filter settings which were adjusted to give maximum information about the various layers. The letter-numbers correspond to the position of the test borings. (The vertical line indicates the maximum depth of the bore hole.) The area between borings D3 and C10 were no bottom reflection is shown was not surveyed because of shoal water.

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14 "Submerged River System of Chesapeake Bay," by J. T. Hack, Bulletin, Geol. Soc. of America, Vol. 68, 1957, pp. 817-830.

15 "Subsurface Stratigraphy of the Atlantic Coastal Plain Between New Jersey and Georgia," by H. G. Richards, Bulletin, Amer. Assoc. of Pet. Geologists, Vol. 29, 1945, pp. 885-955.

Four of the 14 transverse cross sections AA', BB', CC', and DD' are shown in Fig. 6. The cross sections selected are in the area of the Baltimore Channel. No survey track was made along those portions of the sections showing no bottom topography.

A number of passes or tracks were made over areas of particular interest at different gain and filter settings. The data shown in Figs. 5 and 6 are composite tracings of the various data obtained. The horizons shown as light lines have reflection times similar to those expected for multiple reflections from shallower layers. The irregular horizons between 10 ft and 50 ft beneath the bottom, below which no deep horizons are observed, are suspected to be the

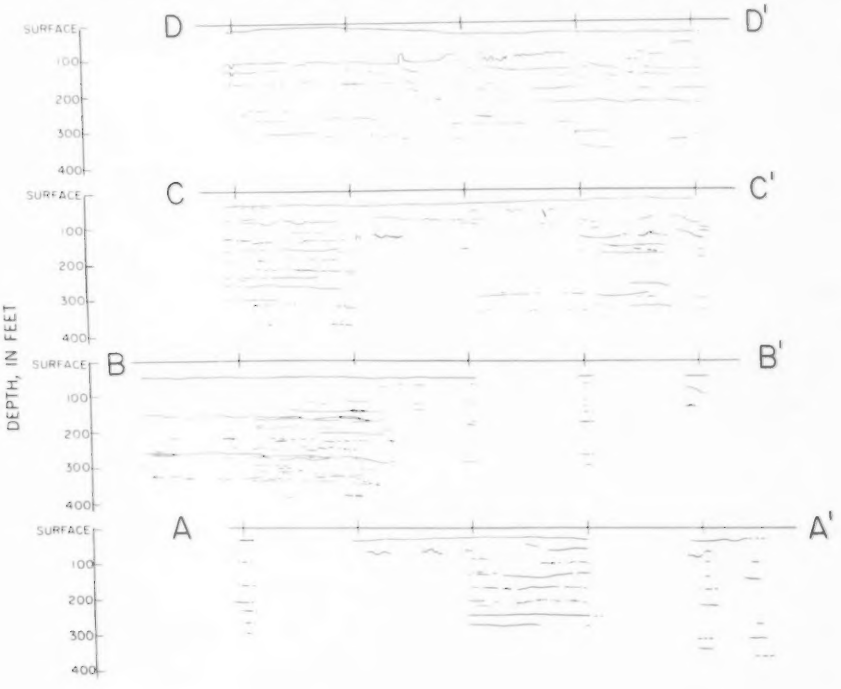

FIG. 6.—TRANSVERSE CROSS SECTIONS

top of gas saturated sediments which have extremely high sound adsorption properties.

The portion of the survey across the Baltimore Channel, along the bridge line, affords a good illustration of the method and of the usefulness of the SDR information. The detailed navigation for this portion is shown in Fig. 3. The positions of cross sections and tracings of records in this area can be seen in Figs. 4, 5 and 6.

Detailed results of the Baltimore Channel along the highway crossing route are presented in Fig. 7. Here tracings of the SDR reflection data[16] superimposed on the boring data[12] is shown. An excellent correlation exists between

16 Alpine Geophysical Assocs., Ind., Norwood, N. J., 1958, "Sub-Bottom Reflection Survey of the Proposed Chesapeake Bay Highway Crossing," Sverdrup and Parcel, Cons. Engrs., St. Louis, Mo., Unpublished report.

the SDR reflection horizons and the water content (porosity) of the layer (shown to the right of the borings) and the resistance to mechanical penetrations as evidenced by the number of hammer blows required to drive a split spoon sampler (shown to the left of the borings). These parameters are indicative of the acoustic impedence (density × velocity) which governs the ability of a layer to reflect sound energy.

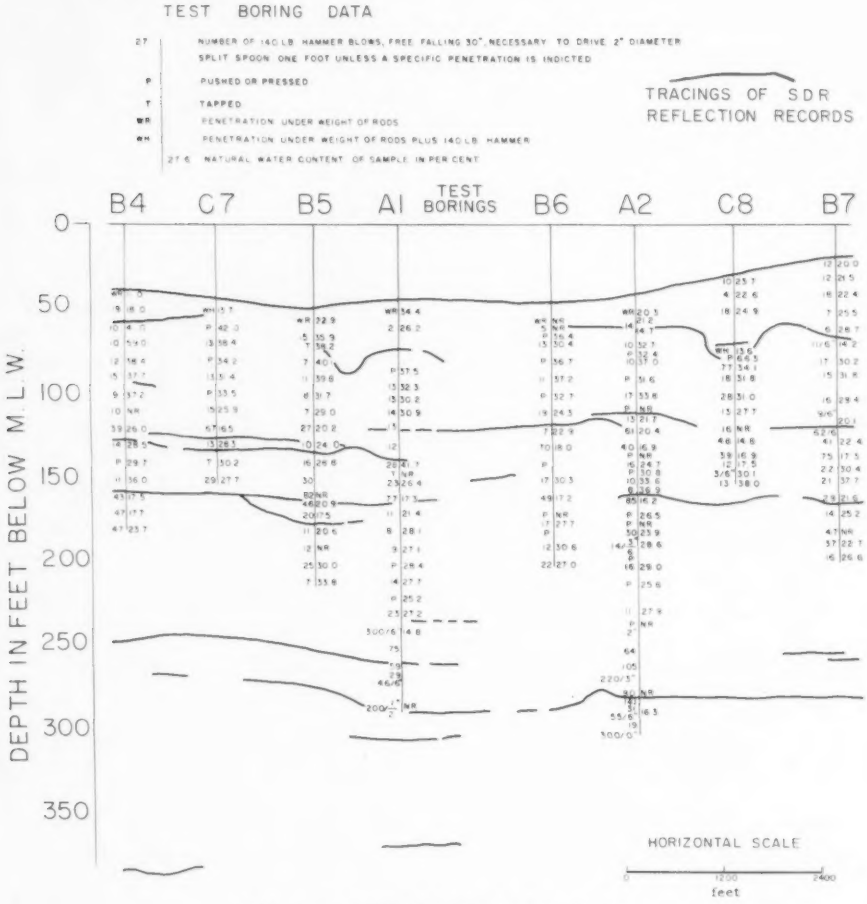

FIG. 7.—BALTIMORE CHANNEL DATA

While the SDR misses some of the minor horizons the major structure is well defined. It affords a continuous record between test borings and gives information on structure beneath the drill holes. The deeper layers were touched, in this case, by holes A1 and A2.

CONCLUSIONS

The Sub-bottom Depth Recorder (SDR) affords a method to map sub-bottom geologic structure in an economical and reliable manner. The data obtained can be correlated with test boring data and affords an excellent horizontal correlation between widely spaced test borings.

The SDR data is useful not only as an adjunct to test borings but provides a valuable guide to the most strategic planning of original or subsequent test drill holes.

A buried river channel corresponding to the old path of the Susquehanna River does not appear to cross the area covered, that is, the mouth of Chesapeake Bay. Some recent shallow channeling was found to exist but is only of minor importance. The Miocene and post-Miocene sediments of the area are flat lying or have very shallow dips.

ACKNOWLEDGMENTS

The authors wish to thank P. Z. Michener of Sverdrup and Parcel, St. Louis, Mo., project engineer for the Chesapeake Bay Highway Crossing, for permission to reproduce the data presented here; the firm of Moran, Proctor, Mueser and Rutledge, New York, N. Y., which furnished the boring data; and the firm of Alpine Geophysical Associates, Inc., Norwood, N. J., which conducted the Sub-bottom Depth Recorder Survey.

Journal of the
SURVEYING AND MAPPING DIVISION
Proceedings of the American Society of Civil Engineers

DISCUSSION

STATUS OF SURVEYING AND MAPPING IN THE UNITED STATES[a]

Discussion by Michael V. Smirnoff, William M. Spann,
Albert A. Stanley, Robert C. Sheldon

MICHAEL V. SMIRNOFF.[1]—The members of the Task Committee must be congratulated on the fine work that they have done in helping to clear up the vital questions of what is professional in surveying and mapping activities, and what functions of such activities must be regarded as engineering. In order to be able to draw such lines of demarkation requires much work, for the accomplishment of which the Committee can hardly be praised enough.

The final report of the Committee leaves no doubt as to what is the proper status of surveying and mapping in the field of engineering, and particularly as to what is the status of property land surveys. The clarification of the question that property land surveys are an integral part of engineering work that the work of a property land surveyor is of professional nature elevates the hard working land surveyor from relative obscurity to professional status.

As in everything else, time brings changes. The records of some old surveys often indicate the description of a real estate as bounded by the lines between some natural monuments, which may be described as an oak tree, a walnot stump, a boulder, etc., and the land as containing approximately so many acres. Not much engineering knowledge was required to make such a survey and to register the property so described in a county office. Such descriptions were definite enough at the time they were made and were acceptable in case of transfer of a real estate from one owner to another.

As the years went by, many natural monuments mentioned in old surveying descriptions became destroyed and one of the main tasks of a property land surveyor became the reestablishment of the old property lines in their original location. Meanwhile, certain agencies of the federal government were doing land surveying of their own, in order to establish national nets of horizontal

Note.—This paper is a part of the copyrighted Journal of the Surveying and Mapping Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. SU 2, July, 1960.

[a] September, 1959, Final Report of the Task Committee on Status of Surveying and Mapping of the Surveying and Mapping Division.

[1] Assoc. Prof. of Civ. Engrg., N. C. State College, Raleigh, N. C.

and vertical controls. Although the work of such agencies is far from being accomplished, it has progressed far enough to offer a permanent frame of reference to which any property survey may be tied. Such a tie assures the permanency of a given land survey, since its lines can be reestablished in their original location, even if all the property survey monuments become destroyed.

Thus, modern property surveys impose many additional requirements on the knowledge that a land surveyor must have. He must be familiar with the United States system of public land surveys and with the state coordinate system established by the United States Coast and Geodetic Survey (USCGC). He must also be familiar with the state and municipal laws governing property surveys in a given region. In addition, he must know how to make stellar and solar observations for finding true directions of surveyed lines. All this requires a certain amount of education that a modern land surveyor must have in order to elevate his work to a professional level.

It is pointed out in the final report of the Task Committee " . . . that in the future it will be imperative that the engineers be very careful, therefore, in judging their own competency to practice property surveying." Thus, in the future, only a necessary qualitative education, or the equivalent amount of land surveying experience, should be considered as prerequisite requirements for licensing surveyors. This should naturally disqualify, from the practice of land surveying, any registered engineer who had neither schooling nor experience in property surveying.

Not so long ago, most engineering schools in the United States required all their engineering students to take at least one semester of elementary surveying. However, at the present time (1960), it is probably true that most engineering schools have eliminated surveying, as a required course, for all but civil engineering students. Some schools reduced surveying instructions to a few laboratory exercises. The question of elevating the work of a land surveyor to a professional level imposes equal requirements for the qualification of engineers for this field of professional activity.

The foregoing situation puts the question of judging the competency of anyone aspiring to do property land surveying work squarely in the hands of state registration boards for professional licensing. Regarding the changes that will need to be introduced in the future in land surveying licensing, it is possible to anticipate two general courses. Either a sufficient number of surveying questions may be included in the state examinations for professional engineers so that the examinees passing those questions may be given professional engineering license in land surveying, or a special surveying examination may be given to those who plan to go into property land surveying activity. This would naturally include the engineers.

The first of the two alternatives may be too drastic a jump for many present-day surveyors, and may be possibly set as an ultimate future goal. The second alternative may work better in a long transitory period of elevating the land surveyor to a professional status.

WILLIAM M. SPANN.[2]—The final report of the Task Committee on the Status of Surveying and Mapping in the United States is a masterpiece of research, completely covering the subject and outlining definite answers for controversial matters. The Committee is commended for its thoroughness of study, its reasonable analysis, and its conclusions.

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[2] Cons. Engr., Member Missouri Bd. of Registration.

There are, of course, some differences of opinion on the status of land surveying and its position in the civil engineering field, with particular reference to the classification of experience records, when being considered as qualification for admission to ASCE, and for registration as professional engineers with state boards. Both are of importance to the person involved in either or both cases, and for that reason the writer wishes to discuss "Reference D, Section 1 A—Property and Boundary Surveys," in which the land surveyor is classified at the "Professional Level," based on his own experience of over 50 yr of activity in land surveying, and some 5 yr in reviewing applications and examining applicants for registration in Missouri as land surveyors.

It appears that the issue concerns the line of demarkation between what is sub-professional and what is professional in classifying the technical qualifying requirements to do the work. Basically, in the writer's opinion at least, surveying has always been a component part of civil engineering. There are two distinct parts to surveying: land surveying, which is defined under reference A of Appendix 1 in the Report, and engineering surveys, which are covered in categories II, III, and IV. The separation of the two classes of surveying does not remove land surveying from civil engineering, but places it in a different class because of its qualifying requirements as to education and experience.

The upgrading of the land surveyor can be accomplished by his being also qualified as a professional engineer. The combined qualifications tend to elevate the position. A professional engineer can be a land surveyor, if qualified to do that type of work, but a land surveyor as such is not qualified to practice engineering.

The basic curriculum at colleges usually limits land surveying subjects to plane or route surveying, which is inadequate to qualify a man to practice without some actual field experience. Many surveyors grow up in the profession and fail to acquire college or technical training in engineering subjects. Such conditions tend to force the need for classifying the simon-pure surveyor as pre-professional in its entirety. Other types of surveys as covered by categories II, II, IV, V, and VI are definitely engineering in classification, and should be recognized as shown on the classification chart (reference D).

The definitions of "professional" and "technician" under section B of the report are good. The only comment the writer can make here is that the wording of the reference to the "technician" places him under a professional person, which in the case of the land surveyor is not usually the case. A registered land surveyor is responsible under his seal for his work, but no professional qualifications are required beyond a knowledge of simple mathematics and use of formulae.

It is granted that the requirements for classification of the land surveyor need up-grading, but that is done when he has the qualifications to advance to the grade of the professional. The writer would like to see the qualifying requirements of ASCE and those for registration in the several states as to professional status equal or uniform in intent to maintain the high standards in admissions to society membership and in the practice of engineering through registration. There are about 37 boards at this time, out of a total of 53, that have laws regulating the practice of land surveying. These laws are variable in detail in that some include some engineering surveys within the scope of registration as a land surveyor. This only serves to broaden the examination requirement by the inclusion of actual engineering survey data that normally would come under professional engineering. It is not difficult to draw the line of demarcation when factual data is shown on an experience record.

It would seem, therefore, that the classification chart (reference D) should be amended by moving "Land Surveyor" under Section 1 A—Property and Boundary Surveys from the column headed "Professional Level" to that headed "Pre-professional Level."

The chart is an excellent guide for those concerned with checking the applicant's qualifications for admission to ASCE, as well as for evaluating the experience records of applicants who want to register with the several state boards of registration.

Because of the far-reaching effects that this report may influence, it seems that the committee should secure the reaction of other groups who are working along similar lines, if it has not been done. The principal ones at this time are the Registration Committee ASCE, the Registration Committee on Land Surveying of NCSBEE, the American Congress on Surveying and Mapping, and the California Council of Civil Engineers and Land Surveyors, and other similar organizations interested in land surveyors. Unity and cooperation are shopworn words, but are very important in the operating procedures of the profession that contains so many splintered groups. Elevating the position of land surveying to a basic place in the civil engineering profession can be accomplished by gradual transition as time goes on, without lowering standards, but it seems best to meet the issue gradually instead of by long range controlled anticipation of possibilities. This can be applied to registration procedures.

ALBERT A. STANLEY.[3]—This report is highly important to the entire profession of surveying and mapping for its careful attention to defining and classifying the various components of surveying and mapping which contribute to the over-all civil applications of engineering. The writer personally applauds any effort that enhances the dignity and standing of the profession of surveying and mapping. This report has contributed immeasurably to this objective.

The first report made in 1955 is valuable for its clarification of the professional and technician levels of surveying. The concise terms developed in the report provide important guides to improved standards for job classifications, especially for those individuals employed by the Federal Government. Definitions of professional positions, given in reference D of the report, are particularly timely and valuable as a basis of comparison. The much needed guidelines which are set forth in the interim report of 1956 will serve to orient surveying into its proper relationship in the field of engineering. The second interim report of 1957 concerning land surveying does not involve functions of the Coast and Geodetic Survey or the Society of American Military Engineers (SAME), with which the writer is particularly concerned, that would require intensive thought on our part. For that reason, the writer's opinion, in this connection, would not have much weight. In passing, the writer would like to indicate, however, agreement with the opinions of those of the civil engineering profession who regard property surveying as an engineering function properly designated as a branch of civil engineering.

No comment is necessary as to geodetic surveying or geodetic engineering, since their place in civil engineering has never been disputed. Cartographic engineering is now receiving the recognition that it has always deserved.

As Chairman of the Membership Committee of the SAME, the writer is especially concerned with engineering fields as they are pertinent to SAME membership classification. This matter has been studied extensively during the

[3] Special Asst. to the Director, U. S. Coast and Geodetic Survey, Washington, D. C.

past year by a special ad hoc Committee on Definitions chaired by Major General Charles G. Holle, C.E. (Retired). It was pointed out in these committee deliberations that the founder engineer societies have been concerned for some time with arriving at a definition of engineering. The latest definition to come to the attention of this committee is that promoted by the Engineers' Council for Professional Development, as follows:

> "Engineering is a profession in which a knowledge of mathematical and natural sciences, gained by study, experience, and practice, is applied with judgment to develop ways to utilize economically the materials and forces of nature for the progressive well-being of man."

We construe military engineering to be any field of engineering that is applied under the direction of military commanders to support, maintain, and increase the effectiveness of military operations. Mention is made of this SAME committee's work as an indication of our thinking in the Society, paralleling in some ways the basic problem inherent in the ASCE committee study—that is, eligibility for membership and proper membership classification. The same philosophy applied to civil functions would certainly indicate a broadening of the concept as to the specific branches of engineering that support civil functions.

The writer is glad that the ASCE is being concerned with the more liberal interpretations, as revealed in the Report on the Status of Surveying and Mapping in the United States. This report is in the interest of the surveying and mapping profession, and the writer is enthusiastic over the progress that has been achieved to date.

ROBERT C. SHELDON,[4] F. ASCE.—This final report and its retention of property surveying as a part of civil engineering is most welcome. The clear definitions contained in the report will be appreciated by all those having problems in this area.

Land Surveying.—The writer is of the opinion that the problem of transition from the land surveyor and professional engineer licenses to the single professional license will not be difficult. It is obvious in those states requiring dual licenses that the legislative bodies have made two basic conclusions, reflected in the laws. These are: that all surveyors are not engineers; and that all engineers are not surveyors. The task committee, and in turn the ASCE, have now, rightly, concluded that all surveyors are, or should be, engineers, and admit that all engineers are not surveyors. One of the basic conclusions of the legislative bodies is still in effect. The writer can see no reason why the boards of Registration, in those states requiring dual license, should not adopt the recommendations of the profession with respect to the status of surveyors and require all persons to qualify for a professional engineers license before admitting them to the examination for surveyor.

Having acted as examiner in surveying for many years for one of the state boards of registration, the writer has been aware of the difficulty in examining candidates for surveyors license. In order to insure that the candidate is qualified in the basic sciences and in engineering surveying, all of which is covered in the professional engineers examination, so much time is used that there is little time to investigate the particular qualities essential to the practice of land surveying. This problem is further complicated by the fact that

[4] Asst. Prof. of Civ. Engrg., Univ. of North Dakota, Grand Forks, N. Dak.

there are few text books, courses, or teachers of land surveying in the educational institutions that the boards of Registration must rely on to supply information.

With all candidates for surveyors license already qualified to practice professional engineering the surveyors examination could be confined to those qualifications peculiar to property surveying. This would insure that the requirements of the law would be met, and that only those qualified to practice would be licensed.

New Umberella Term.—The task committee has recommended three new names, which this writer assumes, are to eventually replace the present name of the Surveying and Mapping Division. The writer was for many years an employee of an organization engaged in property surveys, engineering surveys, geodetic surveys, and cartographic surveys (The Section of Surveys of The Panama Canal). In describing the work in which this organization was engaged, the term of field engineering was found to be very convenient and adequately descriptive. It is the writer's opinion that the title the "Field Engineering Division" would sound well and would describe the relation of the division with the other divisions of the Society.

Education.—In recommending that at least one engineering school should offer graduate degree programs in the surveying engineering field, the task committee has been too conservative. This recommendation is the same as if some similar committee of the hydraulic division would recommend that hydraulic research be confined to one institution. This recommendation will be of little help to those that are trying to correct the serious shortages in surveying education.

SELF-CHECKING METHOD OF COMPUTING CURVE ELEVATIONS[a]

Discussion by T. F. Hickerson

T. F. HICKERSON,[2] M. ASCE.—Mr. Lamont's method of computing curve elevations depends upon the fact that his Eq. 1, $R = \frac{G_2 - G_1}{L}$; representing the rate of change of slope along the parabola is a constant.

The procedure for locating the turning points of the curve appear to be by trial. Also, the work involved in determining elevations at all the stations along the curve appears to be lengthy even by the aid of the type of desk calculator mentioned.

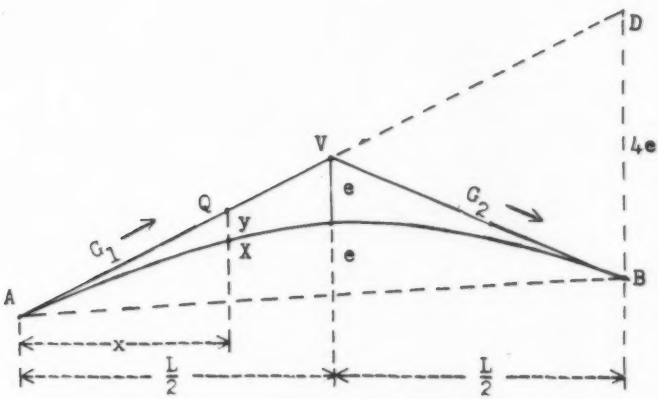

FIG. 3.—COMPUTING CURVE ELEVATIONS

For purposes of comparison, let E_a, E_q, and E_x equal the elevations at points A, Q, and X along the curve (Fig. 3). Then

$$E_x = E_q + y = E_a + G_1 x + y \quad \ldots\ldots\ldots\ldots \quad (4)$$

where the offset y from tangent AD to curve is treated algebraically (being negative for convex curves and positive for concave curves).

Since $\frac{y}{4e} = \left(\frac{x}{L}\right)^2$ and $4e = DB = (G_2 - G_1)\left(\frac{L}{2}\right)$ = vertical divergence of the

[a] February, 1960, by Charles M. Lamont.
[2] Formerly Prof. of Civ. Engrg., Univ. of N. C., Chapel Hill, N. C.

two grade tangents V D and V B, we get $y = \frac{1}{2} R x^2$. Then Eq. 4 becomes

$$E_x = E_a + G_1 x + \frac{1}{2} R x^2 = E_a + (x)\left(G_1 + \frac{1}{2} R x\right) \ldots\ldots (5)$$

Differentiating Eq. 5 and equating to zero for a max. or min., yields $x = -G_1/R = -\frac{0.75}{-0.4} = 1.875$ sta. from A to H.P.

Eq. 5 provides a simple method for obtaining all the desired elevations along the curve. Since the factor $\left(G_1 + \frac{1}{2} R x\right)$ is linear, its value is found easily.

TABLE 2

Station	x (Stations)	(0.75-0.2x)	(x) (0.75-0.2x)	Elev. on Curve.
86 + 75 PC	0	0.75	0	45.630
87	0.25	0.70	0.175	45.805
88	1.25	0.50	0.625	46.255
88 + 62.5 HP	1.875	0.375	0.703	46.333
89	2.25	0.30	0.670	46.300
90	3.25	0.10	0.325	45.955
91	4.25	-0.10	-0.425	45.205
92	5.25	-0.30	-1.575	44.055
93	6.25	-0.50	-3.125	42.505
94	7.25	-0.70	-5.075	40.555
94 + 75 PT	8.00	-0.85	-6.800	38.830

Merely add (algebraically) in succession the product of the factors to E_a, the elevation of the PC, and the result is E_x, checking with the PT at B. The entire curve is referred to only one tangent instead of several tangents. The data of Mr. Lamont's illustrative problem leads to the information shown on Table 2.

INTERSECTION OF STRAIGHT LINE WITH SPIRAL[a]

Discussion by Francis Bates

FRANCIS BATES,[2] F. ASCE.—Mr. Hickerson has developed an intricate procedure for solving a relatively very simple problem. All that is really needed is the computation of a few triangles, that may easily be solved with a desk calculator. For each intersection, the final triangle will first have a tentative bearing for the side that is also the spiral chord, but this may be so closely estimated that repetitive calculation is seldom needed.

This discussion will use simple, flexible "spiral formulas," developed in this particular form by the writer and Mr. Murray Mills,[3] before 1910.

1) The end of each chord, C, (of any desired length), is stationed successively, 1 - 2 - 3 - . . . , indicating the distance, L, from the PS, at which curve of spiral reaches a, 1° - 2° - 3° - . . . curve. The final station, (n), is the degree, D, of the main curve.

2) All angles, including the spiral angle, A, are expressed in minutes.

3) Transposing the author's "well known," Eq. 1(a):

$$A = \frac{60\ D\ L}{200} = \frac{60\ (n)\ (nC)}{200} = (3)\ C/10\ (n)^2 \quad \ldots\ldots\ldots (33)$$

4) The bearing, d', of any chord, Sta. G to Sta. H: With reference to the x-axis:

$$d' = C/10\ \left(H^2 + G^2 + H\ G\right) \quad \ldots\ldots\ldots\ldots\ldots (34)$$

5) Angle, tangent at Sta. G, with the forward chord:

$$C/10\ \left(H^2 - 2\ G^2 + H\ G\right) \quad \ldots\ldots\ldots\ldots\ldots (35)$$

6) Angle, tangent at Sta. H, with the back chord:

$$C/10\ \left(2\ H^2 - G^2 - H\ G\right) \quad \ldots\ldots\ldots\ldots\ldots (36)$$

7) Angle difference, Eq. 36 and Eq. 35:

$$C/10\ (H - G)^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots (37)$$

In the author's example, $n = D = \frac{(9)\ (200)}{300} = 6$; $C = \frac{300}{6} = 50$ ft. Applying the chord length to the sin and cos of successive angles indicated by Eq. 34: (that is: 0° - 05': 0° - 35': and, 1° - 35'): At Sta. 3: $y_3 = 1.9633$; $x_3 = 149.9783$; A_3 = spiral angle = $3\ (50/10)\ (3)^2 = 135' = 2° - 15' - 00''$.

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[a] February, 1960, by T. F. Hickerson.
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Distance to intersection, (a): $160.00 - x_3 = 10.0217$.

Referring to Fig. 1 the radial line, (bearing 87° - 45' - 00'', with reference to the x-axis), is drawn through Sta. 3, a distance of 15 ft to outer Sta. 3_o and inner Sta. 3_i, respectively. All three stations are joined to intersection, a, in the x-axis. Also, join Sta. 3, Sta. 3_o and Sta. 3_i, to points, P_1 - P_2, and P_3, respectively. The latter are intersection points in the "straight line," with

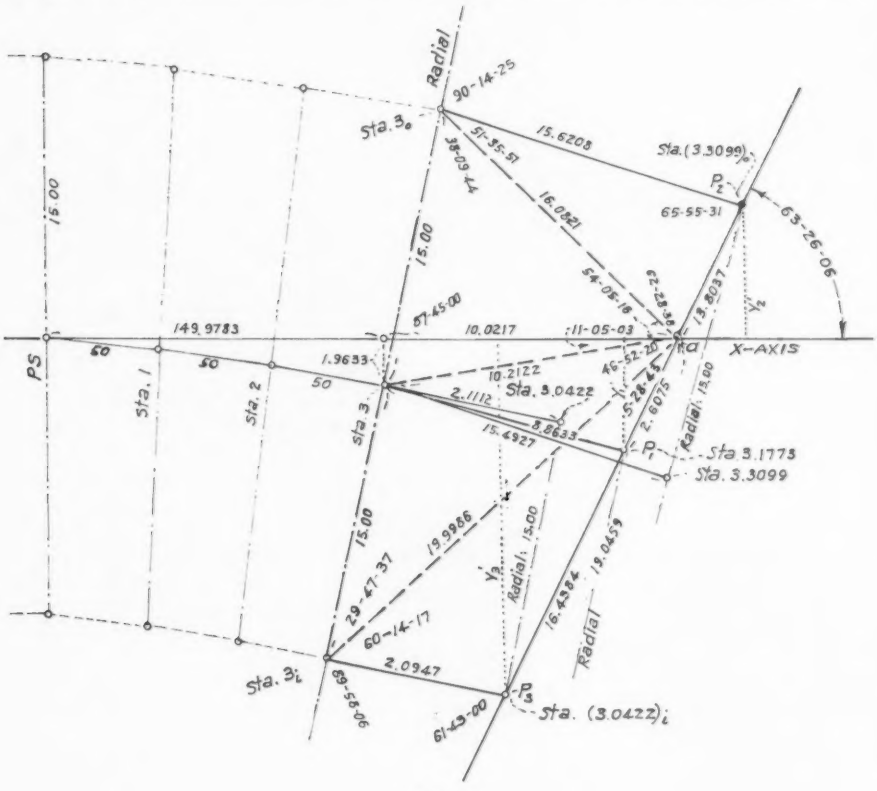

FIG. 2

the respective spiral curves. Basing all bearings with reference to the x-axis, the calls on the former are:

Line (3 - a):	11° - 05' - 03'':	10.2122 ft
Line (3_o - a):	54° - 05' - 16'':	16.0821 ft
Line (3_i - a):	57° - 57' - 23'':	19.9986 ft

These calls are calculated by traverse, based on the x-axis, rather than by triangle solution, in order to avoid subsequent repetitive computation. For example:

		x-axis	y-axis
(3_o - a):	15.0000 cos - sin 87 - 45 - 00:	- 0.58890	- 14.98844
	10.2122 cos - sin 11 - 05 - 03:	+ 10.02170	+ 1.96330
		+ 9.43280	- 13.02514

For line $(3_i - a)$, the signs on the radius traverse are reversed, the corresponding values being + 10.6106 for the x-axis, and + 16.9517 for the y-axis.

The three key triangles, (Sta. 3 - P_1 - a); (Sta. 3_o - P_2 - a); and (Sta. 3_i - P_3 - a) are alike, in that:

1. The intersection point, a, is common to all.
2. One side is the fractional-chord of intersecting spiral.
3. The fixed angle, at point, a, is opposite the chord, and its value is known.
4. One side is in the intersecting "straight line."
5. The length of the third side is known.

The bearing and length of the fractional-chord must satisfy the requirement of both the spiral and the triangle. Since it is opposite the known angle, the chord length will be the more stable, and, therefore, more suitable for the preliminary assumption.

CENTER LINE INTERSECTION

(x-axis): Sta. 3 to point, a: + 10.0217 + 9.0400
(1.9633) $(0.5)^4$ - 0.9817 - 0.1776
+ 8.8624

(9.0400) (tan 2 - 15 - 00)[5] = 0.3552: @ $(0.5)^4$
Estimated length of intersecting chord:
Estimated L_1' = 158.8624 ft = 3.17725 chords.
Testing this length in the spiral, for chord bearing:

$50/10 \left[(3.17725)^2 + (3.00)^2 + (3.17725)(3.00)\right] = 143.13334 = 2° - 23' - 08''$:

And, testing this bearing in the triangle:

$\frac{10.2122}{\sin 65 - 49 - 14}$ = 11.19431: @ sin 13 - 28 - 11 = 2.6075: Line (a - P_1)
11.19431: @ sin 52 - 21 - 03 = 8.8633: Line (3 - P_1)
Initial assumption: (8.8624)

2.6075 sin 63 - 26 - 06 = 2.3322 = y_1': 160.0000
2.6075 sin 26 - 33 - 54 = 1.1661 - 1.1661 158.8339 = x_1':

$A_1' = 50/10\ (3.17727)^2\ (3) = 151.4257 = 2° - 31' - 25.5''$: $L_1' = 158.8633$:

OUTER SPIRAL INTERSECTION

The other intersections are found in a similar manner. It would be unrewarding to continue if it were not for the difference in length between the offset chord and its mate. The former appears in the triangle, while its adjusted length, as projected to the center-line chord, must be used for finding the bearing.

(x-axis): Sta. 3_o to point, a: ------- + 9.4328
(13.0252) $(0.5)^4$ --------- + 6.5126 + 15.9454
(15.9454) (tan 2 - 15 - 00)[5] = 0.6261: $(0.5)^4$ - 0.3132
+ 15.6322

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[4] Contangent, 63°-26'-06".
[5] Bearing of spiral tangent, at Sta. 3.

(Preliminary: 165.6322 = 3.3126 chords;)

Eq. 35: $50/10\left[(3.3126)^2 - 2(3)^2 + (3.3126)\,(3)\right] = 14.5556$: 0° - 14' - 33''.

Eq. 37: $50/10\,(0.3126)^2 = 0.4886$ min 0° - 00' - 29''.

15 (sin 0 - 14 - 33 + sin 0 - 15 - 02) = 0.12915. - 0.12915: 0° - 15' - 02''.

+ 15.63220:

165.50305 ft = 3.31007 chords: (center line) + 15.50305

Testing in the center-line spiral, for chord bearing:

$50/10\left[(3.31007)^2 + (3.00)^2 + (3.31007)\,(3.00)\right] = 149.4339$: 2° - 29' - 26''

Tangent to spiral, at Sta. 3: 2° - 15' - 00''

0° - 14' - 26''

Eq. 37: $50/10\,(0.31007)^2 = 0.4807$ ft ---------- 0° - 00' - 29''

0° - 14' - 55''

15 (sin 0 - 14 - 26 + sin 0 - 14 - 55) = - 0.12806

$\frac{16.0826}{\sin 65 - 55 - 32} = 17.61421$: @ sin 62 - 28 - 38 = + 15.62076 + 15.49270

17.61421: @ sin 51 - 35 - 51 = 13.8037

$L_2' = 165.49270 = 3.309854$ center-line chords.

13.8037 sin 63 - 26 - 06 = 12.3464 = y_2'. 160.0000

13.8037 sin 26 - 33 - 54 = 6.1732: + 6.1732 166.1732 = x_2'.

$A_2' = 50/10\,(3.30985)^2\,(3) = 164.3266$ ft = 2° - 44' - 20''.

INNER SPIRAL INTERSECTION

(x-axis): Sta. 3_i to point, a: + 10.61060

(16.9517) (0.5) - 8.47585 + 2.13475

2.13475 (tan 2 - 15 - 00) = 0.08388: @ (0.5) - 0.04194 + 2.09281

(Preliminary): 152.09281 = 3.041856 chords.

$50/10\left[(3.0419)^2 - (2)\,(3.00)^2 + (3.0419)\,(3.00)\right] = 1.8909$: 0° - 01' - 53.5''

$50/10\left[(2)\,(3.0419)^2 - (3.00)^2 - (3.0419)\,(3.00)\right] = 1.9031$: 0° - 01' - 54''

15 (sin 0 - 01 - 53.5 + sin 0- 01 - 54) = 0.01651

+ 2.09281 2.10932

Estimated, on center-line: 152.10932 = 3.042186 chords

Test this in center-line spiral, for chord bearing:

$50/10\left[(3.04219)^2 + (3.00)^2 + (3.04219)\,(3.00)\right] = 136.9074$: 2° - 16' - 54'':

And test this bearing in the triangle, for inner chord length:

$\frac{19.99864}{\sin 65 - 43 - 00} = 21.93987$: @ sin 60 - 14 - 17 = 19.04585:

21.93987: @ sin 5 - 28 - 43 = + 2.09468

$L_3' = 152.1112 = 3.04222$ chords. (center-line) + 0.01651

+ 2.11119

19.04585 sin 63 - 26 - 06 = 17.0351 = y_3'. 160.0000

19.04585 sin 26 - 33 - 54 = 8.5176: - 8.5176 151.4824 = x_3'.

$A_3' = 50/10\,(3.04222)^2\,(3) = 138.82654 = 2° - 18' - 50''$.

The results obtained here are practically identical with those of the author. They could be carried to any degree of refinement by retaining more decimal places in the process. In each instance, the first approximation for length of

the intersecting chord agrees, within two decimal places, with the final determination. In addition, repetitive computation is unnecessary.

Such accurate approximation is not essential, but otherwise could result in more trials before the requirement of the spiral and triangle are simultaneously satisfied. The key to accurate approximation, however, is to keep the base of operations, (such as Sta. 3), fairly close to the intersections. The base for each intersection can be shifted if the angle of intersection is more acute.

ERRATA

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p. 11. The radical sign in the expression for d below Fig. 3 should be extended so that it encompasses the term ΔX^2.

p. 28. In Fundamental Operation 5 a vertical rule should be drawn to the right of the letters P, C, and K in the first column.

p. 28. In Fundamental Operation 6 in the fifth part of this table put a vertical rule to the right of the word DIVIDE and to the left of the letter q.

p. 29. Under the heading "Intersection of a Circular Curve with a Straight Line" the word "See" should be deleted.

p. 33. Under the heading "Intersection of Two Circular Curves" in item 1 a horizontal rule should be drawn over the term $C_1\ C_2$. In item 2 parentheses should be inserted to encompass the terms $r_1^2 - r_2^2 + d^2$.

p. 33. Under the heading "Circular Curve Joining Two Given Circular Curves" in item 1 change "in paragraph f" to "in the preceding paragraph."

p. 35. Under the heading "A Circular Curve Joining a Tangent at a Given Point in Passing Through a Given Point" at the end of the item entitled "Given" insert a period.

p. 38. In item 8 change the term "$C_1\ C_\omega\ H$" to "$C_1\ C_2\ H$."

p. 40. In item 7 put a horizontal rule over the terms "C_1P" and "C_2P."

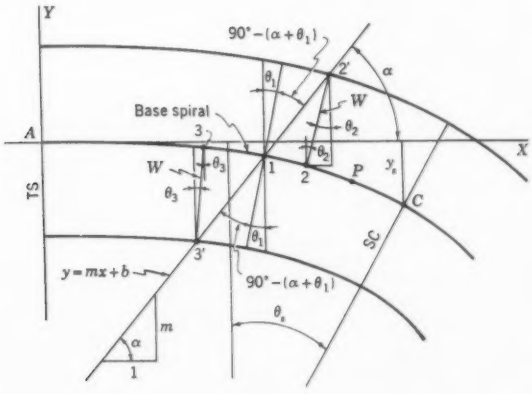
p. 40. Change item 1 to read

1. Compute radius r_1 and coordinates of point C_1, as outlined in application "A Circular Curve Joining a Tangent at a Given Point and Passing Through a Given Point."

p. 40. In item 3, put a horizontal rule above the terms V_2B and V_2P.

p. 62. In Eq. 2b the term $\theta^3/12$ should be changed to $\theta^3/42$.

p. 62. The drawing identified as Fig. 1 should be moved to p. 66 and be entitled Fig. 2. Fig. 1 should be the following:



p. 64. In Eq. 15 the term $\theta^3/12$ should be changed to $\theta^3/42$.

p. 64. Eq. 18 should be corrected to

$$K\left\{\left(l_s/\sqrt{\theta_s}\right)\left[\frac{1}{2} - \frac{1}{12}\,\theta_2^2 + \frac{m}{\theta_2}\left(1/2 - 1/4\,\theta_2^2 + \frac{1}{48}\,\theta_2^4\right)\right] + \left(W/\sqrt{\theta_2}\right)\left(\sin\theta_2\right)\right.$$

$$\left. + m\cos\theta_2\right\} = \left(l_s/\sqrt{\theta_s}\right)\left[1/3\,\theta_2 - \frac{1}{12}\,\theta_2^3 + \frac{1}{1320}\,\theta_2^5 + m\left(1 - \frac{1}{10}\,\theta_2^2\right.\right.$$

$$\left.\left. + \frac{1}{216}\,\theta_2^4 - \frac{1}{9360}\,\theta_2^6\right)\right] - \left(1/\sqrt{\theta_2}\right)\,W\left[\left(\cos\theta_2 - m\sin\theta_2\right) + m\,a\right] \quad \ldots\ldots(18)$$

p. 64. The equation of C_1 should be identified as Eq. 19.

p. 64. In Eq. 20 a bracket should be inserted just after the term l_s.

p. 65. Eq. 27 should be corrected to

$$y_s = l_s\left(\frac{1}{3}\;\theta_s\right)\left(\frac{\pi}{180}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots(27)$$

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PROCEEDINGS PAPERS

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c. Discussion of several papers, grouped by divisions.

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c. Discussion of several papers, grouped by divisions.

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PART 2

JULY 1960 – 22 NO. SU 2
VOLUME 86 PART 2

Your attention is invited

NEWS OF THE SURVEYING AND MAPPING DIVISION OF ASCE

JOURNAL OF THE SURVEYING AND MAPPING DIVISION
PROCEEDINGS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

DIVISION ACTIVITIES

SURVEYING AND MAPPING DIVISION

Proceedings of the American Society of Civil Engineers

NEWS

July, 1960

Meetings: I. S. P. (International Society for Photogrammetry), Ninth International Congress of Photogrammetry, London, England, September 5-21,1960

I. A. G. (International Association of Geodesy) Twelfth General Assembly, I.U.G.G., Helsinki, Finland, July 25-Aug. 6, 1960

RENO CONVENTION (June 19-23, 1960)

Mr. D. H. Rutledge, Topographic Division, U.S.G.S., at Menlo Park, California was the local chairman in charge of program arrangements for the surveying and mapping sessions at the Reno Convention. The sessions were held on Monday morning and afternoon and on Friday morning.

Monday:

1. "Geodetic Surveys to Measure Earth Movements in Areas of Seismic Activity"

   Capt. William F. Malnate, U. S. Coast and Geodetic Survey

   Describes the history of the measurement program and reviews the horizontal and/or vertical displacement in the El Centro and Tehachapi-Bakersfield area, California; in the Dixie Valley, Nevada; and in the more recent Montana area.

2. "New Methods in Cadastral Surveying"

   Franklin K. VanZandt, Bureau of Land Management

   Covers electronic distance measurement as well as photogrammetric applications to cadastral surveys.

3. "Short Cuts in Location, Right of Way Descriptions, and Design of Road Through Use of Photogrammetry and Electronic-Computation"

   Charles O. Greenwood, Consulting Engineer, Sacramento

4. "An Unusual Land Subdivision Project"

   Alton H. Sorenson, Consulting Engineer, Salt Lake City

   Describes the subdivision of the town of Wendover, Utah, many years after the houses were built. The town was established by the Western

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Pacific on company land. When the railroad decided to sell the individual houses, it was necessary to subdivide the land accurately along the lines established by haphazard fences tacitly accepted by the families living in the houses.

5. "Field Control for Antarctic Mapping"
   William H. Chapman, U.S.G.S.
   Describes experiences of two tours of duty in Antarctica engaged in control surveys (Illustrated).

6. "Large Scale Mapping of Lake Powell"
   Clyde D. Gessel, U. S. Bureau of Reclamation, and Dwight H. Rutledge, U. S. Geological Survey.
   Lake Powell is the reservoir being formed by Glen Canyon Dam in picturesque Colorado Plateau Area. Mr. Gessel is responsible for procurement of the map and Mr. Rutledge is responsible for the control surveys.

Friday morning: (Joint Session with Highway Division)

Panel Discussion on the "Problems Associated with the Professional Engineer and Contracts for Topographic Mapping."
Larry L. Funk, California Division of Highways
Henderson McGee, U. S. Corps of Engineers
Clair A. Hill, Consulting Engineer and member of State Board of Registration for Professional Engineers

ENGINEERING EDUCATION

Program for
Surveying and Mapping session, Civil Engineering Division
American Society for Engineering Education
Annual Meeting at
Purdue University, Lafayette, Indiana

Friday Morning, June 24, 1960

CIVIL ENGINEERING

1. "The Teaching of Surveying Today"
   R. C. Rautenstraus, University of Colorado

2. "The Place of Geodesy in the Required Surveying Course for Civil Engineers"
   Panel: Max O. Laird, Western Electric Co., New York City
   Lansing G. Simmons, Coast and Geodetic Survey, Washington, D. C.
   Carl I. Aslakson, Geonautics, Inc., Washington, D. C.
   Ken S. Curtis, Purdue University, Lafayette, Ind.

3. "The Place of Photogrammetry in the Required Surveying Course for Civil Engineers"
   Panel: Al O. Quinn, Aero Service Corp., Philadelphia
   R. H. Sheik, Ohio Department of Highways, Columbus
   Robert S. Scott, Scott Engineering Co., Alpena, Michigan
   John O. Eichler, Georgia Institute of Technology, Atlanta, Ga.

EXECUTIVE COMMITTEE MEETING

The Executive Committee of the Surveying and Mapping Division met in Washington, D. C., on Wednesday, March 23, and Friday, March 25. The activities of the Technical Committees were reviewed during the meeting and reports were received on program arrangements for technical sessions at future conventions.

The organizational structure of the Division was reviewed in light of progress made since the meeting at Blackduck, Minnesota. It was agreed that further consideration should be given to the wording of the purpose of the Division's responsibility in the field of interplanetary surveys, and the surveys related to oceanography. It was further agreed that the purposes of the Technical Committees of the Division should be re-examined with the foregoing fields of interplanetary surveys and oceanographic surveys in mind.

The Executive Committee considered future activity in regard to implementing the recommendations of the Task Committee on Status of Surveying and Mapping, which were approved by the Board of Direction in February, 1959. In this connection considerable attention was paid to the field of activity of the recently created Task Subcommittee on Professional Practice of Surveying and Mapping. It was noted that the Task Subcommittee held its first meeting in Washington, D. C., on March 25. It was agreed that close liaison and full cooperation betweeen the Executive Committee and this Task Subcommittee was highly desirable.

Ways and means of establishing a Division Prize for the best paper, prepared by a student, on surveying and mapping subjects were discussed. It was agreed that the Executive Committee members should explore means of financing such a prize and that the matter would be taken up at the next Executive Committee meeting.

It was agreed that consideration should be given to establishing terms of office for members of Technical Committees. This matter is to be reviewed by Executive Committee members with a view toward final action at the next meeting of the Executive Committee.

Professor Arthur J. McNair, Vice Chairman of the Executive Committee, invited the committee to hold its next meeting at the Summer Surveying Camp of Cornell University near Ithaca, New York. This invitation was gratefully accepted, and it was tentatively agreed to meet about the middle of August.

TASK SUBCOMMITTEE ON PROFESSIONAL PRACTICE OF SURVEYING AND MAPPING AUTHORIZED

The February issue of the Newsletter of the Surveying and Mapping Division contained an item regarding the proposed establishment of a new committee to implement the action taken by the Board of Direction in regard to the place of surveying and mapping in the civil engineering profession. After a thorough discussion of this matter with officers of the Society, it was decided to create a Task Subcommittee on Professional Practice of Surveying and Mapping, of the Committee on Professional Practice of the Society. The Chairman of this Task Subcommittee is Edmund Friedman, and its members are: Brother B. Austin Barry, Messrs. Geo D. Whitmore and Alfred O. Quinn.

The Task Subcommittee held its first meeting on March 25 in Washington.

Mr. Donald D. King, Assistant to the Secretary, and Director Daniel B. Ventres also attended the meeting.

At this meeting objectives were outlined, and means of publicizing the professional status of surveying and mapping were discussed. Cognizance was taken of the increased workload that might be placed upon the Professional Conduct Committee of the Society for the next few months or years because of the newly declared position of surveying and mapping in the field of professional engineering.

Consideration was given to establishing a date at which time it would be expected that all parties concerned would be operating in the field of surveying and mapping according to the ethical Code of the American Society of Civil Engineers.

The work of this Task Subcommittee will be of major interest to those members of the Society concerned with surveying and mapping.

SURVEYING AND MAPPING TECHNICAL SESSIONS AT NEW ORLEANS CONVENTION

Two technical sessions of the Surveying and Mapping Division were held at the New Orleans Convention on March 11. Due largely to the special efforts made by the local chairman in charge of program arrangements, Mr. Joseph J. Krebs, Jr., the sessions were very well attended despite the lack of advance publicity in "Civil Engineering."

Excellent papers on surveying and mapping problems along the Gulf Coast were presented as follows:

1. "The History of the Surveying and Mapping Division"
   Earle J. Fennell, Chairman, Executive Committee
2. "Offshore Surveying in Louisiana"
   Mr. George Jones, Survey Engineer, The California Company
3. "Low-Water Line Surveys of the Mississippi Delta"
   Clarence R. Reed, Cmdr. Coast & Geodetic Survey, New Orleans District Officer
4. "Location and Construction Survey for the Greater New Orleans Causeway"
   James M. Tuttle, Civil Engineer, Palmer & Baker Engineering, Inc.
5. "The Use of the Tellurometer in Southern Louisiana"
   Coleman Kuhn, Partner, Gandolfo, Kuhn and Walker, Engineers and Surveyors.

ELECTRONIC DISTANCE MEASURING DEVICES

Roland H. Moore, Chairman, Committee on Research and Development, Surveying & Mapping Division

Engineers need no longer budget as much time or money for the work of obtaining reliable geodetic control. They can now choose among three reasonably priced portable electronic distance-measuring instruments which can measure lines with geodetic accuracy in a small fraction of the time previously required for conventional taping. All three instruments are similar in that they measure distances by means of highly stable modulating frequencies superimposed on a carrier wave of much higher frequency. These instruments are the Geodimeter, the Tellurometer, and the Micro-Dist.

The first instrument of this type to be perfected was the Geodimeter, developed in Sweden by Dr. Eric Bergstrand. For the carrier wave of the Geodimeter, Dr. Bergstrand chose visible light emitted by a small incandescent bulb. The light beam is cross-polarized and then modulated by one of several measuring frequencies before it emerges from the transmitting optics; the modulation chops the beam into a continuous series of very-short duration, unifromly-spaced pulses. At the far end of the line, a reflector returns the light to the receiving optics, which pass it to a balance circuit where the phase of the returned pulses is compared with that of the emitted pulses. Phase differences between the transmitted and received light are indicated on an electrical balance or null indicator. Measurement readings are taken on a graduated dial after phase balance has been restored by introducing a measurable segment of a simulated light wave—which in effect changes the length of the return path of the light beam. Readings are taken on several preselected modulation frequencies to provide the data needed for calculating the number of wave lengths and the distance between the Geodimeter and the reflector. Obstacles near but not directly on the sightline do not affect the measuring process.

Three models of the Geodimeter are available. The large Models I, II, and IIA are considered the most accurate geodetic distance-measuring instruments available. Lines 40 miles long or longer can be measured within the limits of accuracy required for first-order base lines. The latest Geodimeter, the Model IV, is a scaled-down version of the larger instruments, which weighs about the same as the other portable instruments described below. The Model IV will measure distances between 50 feet and 5 miles with high precision. Darkness is required for measuring long lines, but measurements up to 0.75 mile can be made in daylight with very little special preparation.

The second distance-measuring instrument to be discussed here is the Tellurometer, which was developed in the Telecommunications Research Laboratory of the South African Council for Scientific and Industrial Research by T. L. Wadley. The original model of the Tellurometer system consists of a master and a remote unit which are stationed at opposite ends of the line to be measured; the units are very similar in appearance but serve different and noninterchangeable functions. The Tellurometer works equally well in daylight or darkness because it uses a very-high-frequency radio wave as a carrier. This carrier wave, centered around 3,000 megacycles and variable by $\pm$ 10 percent, is modulated by preselected measuring frequencies. Communication between units is provided by built-in telephones.

In the Tellurometer system, all measuring data are taken at the master unit, from patterns displayed on a cathode-ray tube. The system actually measures the time, in millimicroseconds, required for a radio signal to travel from the master unit to the remote unit and back. The slope distance between units is obtained by multiplying half the observed transit time by the known velocity of radio waves. The stated accuracy of the Tellurometer in measuring distances between 1500 feet and 40 miles is 3 parts per million + 2 inches. The absolute value of 2 inches results from internal errors of the instruments and from the minimum resolvable distance; it is supposedly unrelated to the magnitude of the distance measured. Some users have found that occasionally the accuracy of short lines does not meet the stated accuracy. Objects like trees or brush in the line of sight tend to disrupt Tellurometer readings, especially if the objects are near either unit and are ruffled

by light wind. Similar obstructions near the midpoint of a long line are much less likely to affect the readings. Although seldom encountered, severe reflections completely disrupt the measuring operation.

Most recently the Cubic Corporation, of California, has introduced the Micro-Dist instrument, which operates in much the same way as the Telluro-meter. Each instrument can operate either as a master or a remote unit. Readings are taken from a calibrated dial instead of a cathode-ray tube. Maintenance problems are reduced by extensive use of transistorized, plug-in circuits. The high carrier frequency permits a very narrow measuring beam without the need for a large reflector. The Micro-Dist is stated to be usable over distances from 50 feet to 50 miles, with an accuracy of 3 parts per million $\pm$ 1 inch.

UNITED NATIONS CARTOGRAPHIC PUBLICATIONS

A number of most interesting publications, mostly dealing with cartographic cooperation among the nations of the world and individual cartographic progress of various nations, are available through the United Nations. (United Nations Sales Agents in the United States is: International Documents Service, Columbia University Press, 2960 Broadway, New York 27, N. Y.)

An article describing the United Nations activity in this field was published in Surveying and Mapping, Vol. 13, No. 4, Oct.-Dec. 1953, p. 509-514. It was written by Dr. Te-Lou Tchang, Chief, Cartographic Section, Bureau of Economic Affairs, United Nations.

Selected List of United Nations Cartographic Publications*
(as of November 1959)

Title of Publication (issued in two editions in English and in French)	Sales Number	Price (in US $)
Modern Cartography, Base Maps for World Needs	1949.I.19	1.00
United Nations Cartographic Bulletin, World Cartography		
Volume I	1951.I.9	1.25
Volume II	1953.II.11	1.25
Volume III	1955.I.30	.60
Volume IV	1956.I.9	.60
Volume V	1957.I.18	.60
Volume VI	-	(in printing)
United Nations Regional Cartographic Conference for Asia and the Far East (Mussoorie, India)		
Volume I Report of the Conference	1955.I.29	.25
Volume II Proceedings of the Conference and Technical Papers	1956.I.23	1.50

* These publications can be purchased from United Nations Sales Agents.

Second United Nations Regional Cartographic Conference for Asia and the Far East (Tokyo, Japan)

Volume I	Report of the Conference	59.I.9	.25
Volume II	Proceedings, etc.	-	(in printing)

International Map of the World on the Millionth Scale**	(1955)	1957.I.29	.80
" " " " " "	(1956)	58.I.2	1.00
" " " " " "	(1957)	58.I.11	1.00
" " " " " "	(1958)	59.I.15	1.00
" " " " " "	(1959)	-	(in preparation)

STORIES OF SURVEYING AND MAPPING FOR THE HIGH SCHOOL STUDENT

Your editor has often wondered if there were any fiction or non-fiction books on Surveying and Mapping which have been written especially for the junior or senior high school age level. Stories of science and adventure fill the bookshelves of our public schools; but, how is our profession represented at this level?

Your editor has been attempting to make a small study of this (as time permits) and it has been most interesting to read the limited number of books covering our field of Surveying and Mapping. Several of these books, with short summaries, are listed here. If any one knows of any other books available, please contact the editor with details. Your editor suggests the possibility of the purchase of the books, the reading of them, and the presentation of them to the local school system or library. Certainly your children would enjoy them and a number of oldsters have found them most interesting.

STEADY AS YOU GO — by Rear Admiral Robert F. A. Studds, Dodd, Mead and Co., New York, 1956, 306 p., $3.00.

Story of a college student's adventures with the U. S. Coast and Geodetic Survey on a summer work assignment and subsequently as a commissioned ensign in the Survey. It is chock-full of adventure as it presents the challenging and satisfying career of a young man as he charts coastal waters and performs triangulation and leveling throughout the United States and its possessions. Author is former Director of the Survey.

HIGH, WIDE AND DEEP — by John J. Floherty, J. P. Lippincott Co., Philadelphia and New York, 1952, 154 p. $2.75.

A career book on the science and adventure in the activities of the U. S. Coast and Geodetic Survey. The nine chapters deal with these interesting activities which most people know little about. The author is known for his many career books.

THE STOLEN SPRUCE — by Kenneth Andler, Ariel Books, New York, 1952, 168 p., $2.50.

A story about a search for an ancient boundary line and a vital longlost corner tree by a boy and his friend deep in the forests of Maine and their struggle with an unscrupulous timber owner and his lumberjacks. Woods

** Annual Progress Report

lore, the art of living in the woods, and the surveying of boundary lines in timberland is taught by their old surveyor friend. The author is a lawyer from New Hampshire who writes from his experiences.

THE SIGNAL NET by Kenneth Andler, Ariel Books, New York, 1953, 183 p., $2.75.

Fast-moving story of the same two boys and by the same author as THE STOLEN SPRUCE. Deals with the kidnapping of an old man by a gang of smugglers and the boys attempt to rescue him. They work with a triangulation party of the U. S. Coast and Geodetic Survey in the woods of Maine and New Hampshire and skillfully interwoven with the plot is the vivid picture of the work of the Survey.

THE MAP MAKER by Kerry Wood, MacMillan, Toronto, 1955, 185 p., $2.75.

The story of David Thompson, one of Canada's great explorer-surveyors who explored, surveyed, and mapped vast stretches of raw land with the Hudson's Bay Company and NorthWest Company. Among his greatest accomplishments were the exploration of the mighty Peace River and the surveying of 49th Parallel to fix the boundary line between the United States and Canada. One of a historical series of Great Stories of Canada for young people.

THERE'S ADVENTURE IN CIVIL ENGINEERING by Neil P. Ruzic, Popular Mechanics Press, Chicago, 1958, 187 p., $2.95.

The story of two boys who spend their summer vacation accompanying a science writer as he gathers information on the construction of the Pan-American Highway and the civil engineering story behind it. Their adventures include contact with highways, construction, structures, foundations, city planning, tunneling, bridges, waterways, surveying, and mapping as they travel from the Arctic Circle to Cape Horn.

MAPPING THE WORLD by Erwin Raisz, Abelard-Schuman Publishers, New York, 1956, 107 p., $3.00.

The fascinating story of map making, from early sand etchings to aerial photographs. The author has succeeded in making a complex subject not easy to understand, but exciting to read about. All the famous names of the past who have played a part in the science of obtaining mother earth's portrait are included. Dr. Raisz has written several books on cartography, but this is his first book written primarily for young people.

DOWN TO EARTH: MAPPING FOR EVERYBODY by David Greenhood, Holiday House, New York, 1951, 262 p., $5.00.

This book is written to be read rather than studied. It is for the amateur, designed to give him an understanding and appreciation of maps, whether his interest be as user, maker, or collector. A most entertaining and instructive book with the theories and techniques of mapping developed in simple language. Probably the best popular presentation of cartography which is available.

FUTURE ASCE CONVENTIONS

October 9-12, 1960	Boston, Massachusetts
April 10-15, 1961	Phoenix, Arizona
October 16-20, 1961	New York City, New York
February 19-23, 1962	Houston, Texas
May 14-18, 1962	Omaha, Nebraska
October 8-12, 1962	Detroit, Michigan
February 25-March 1, 1960	Atlanta, Georgia

COMBINED INDEX TO ASCE PUBLICATIONS

For complete coverage of the Society's 1959 year in print, there is now a Combined Index covering the Division Journals, Transactions, and Civil Engineering. Also included are reprints of the Proceedings Abstracts that are published each month in Civil Engineering. The price of the Combined Index (ASCE publication 1960-10) is $2.00 with the usual 50% discount to members. The coupon herewith will make ordering easy:

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USE OF SURVEYING AND MAPPING DIVISION NEWSLETTER

You are urged to make use of this Division Newsletter for announcements, inquiries, personnel news, committee reports, surveys, and other items of interest to Division members. Please send these items to the editor:

Professor Kenneth S. Curtis
School of Civil Engineering
Purdue University
Lafayette, Indiana